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April 19, 1934

Winter Concreting of Rapide Croche Lock

Large Retaining Wall Tests—V

Defective Design Caused Elevated Tank Collapse

Bearing Capacity of Earth Foundations

Corrugated Metal Arch Widens Concrete Bridge

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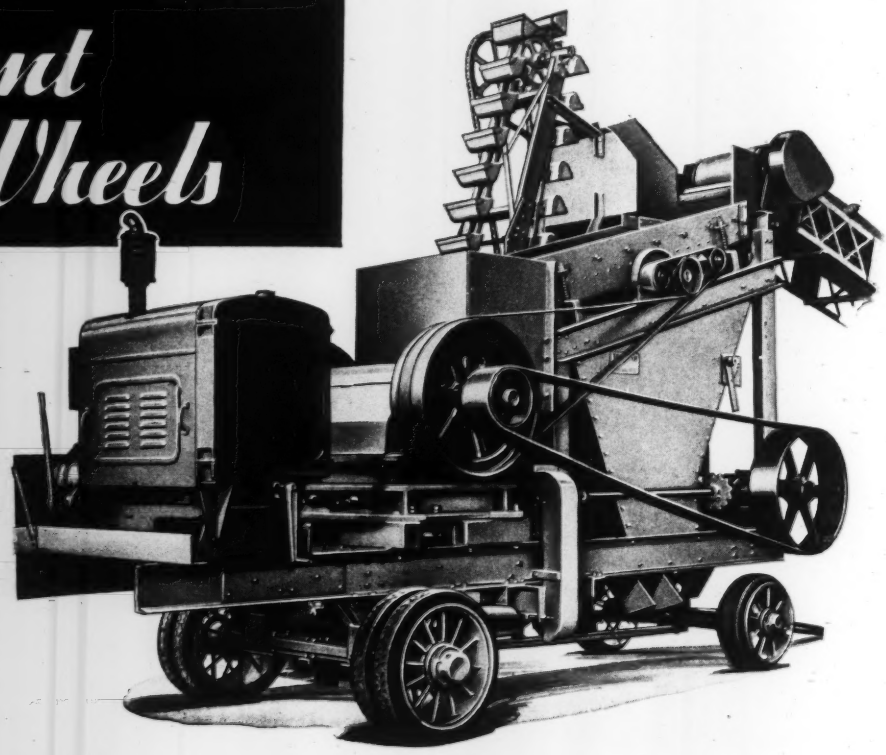

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In the News:

THE DIVERSION TUNNELS for the Fort Peck Dam project in Montana will be built for 7⅓ million dollars under a bid submitted by Silas Mason, Inc., New York, and the Walsh Construction Co., Davenport, Iowa.

FIRST EXERCISE of the PWA's right to condemn land for low-cost housing is being carried out in Atlanta, Ga., in connection with the University housing project for negroes. Precedent is being set for similar activities in other cities, to expedite the program of the Public Works Emergency Housing Corp.

A NEW PUBLIC-WORKS BILL introduced by Senator LaFollette differs from the original NIRA in three main particulars —it provides for three times as large an appropriation (for a three-year program, it is understood), it liberalizes the loan-security and grant-percentage requirements, and it carefully provides for and emphasizes non-federal projects. Grade crossing elimination is specifically urged. Pending determination of the method by which the bill will be offered to Congress, it has been referred to the Senate committee on education and labor.

CLEVELAND'S TROUBLES, arising from an empty treasury, continue to multiply, with threats to dump garbage in the street if it is not collected and rumors of no pay for city employees as typical. At the special election on May 15, to approve an issue of deficiency bonds, candles instead of electric lights will be used in the voting booths.

THE FIRST ANNIVERSARY of the CCC was marked by an enthusiastic report to the President by Robert Fechner, director. Some 600,000 men have been enrolled, much useful work done and, of $255,000,000 disbursed, about $60,000,000 was sent home by the men.

In This Issue:

MOBILE CONSTRUCTION PLANT and plenty of steam heat for aggregate piles, mixing water and batcher bins were the essential factors that contributed to the successful completion of a new lock at Rapide Croche on the Lower Fox River in Wisconsin. An article on the work comprises an excellent account of modern winter construction.

A PRACTICAL EXERCISE in applying soil-mechanics theory, by reasoning from a specific case of measured soil-bearing capacity to a possible method of determining the capacity by laboratory tests, is presented by a case from Torrance, Calif.

CONCRETE PAVING in sub-zero weather carried on by the Ohio state highway department last winter included the use of high-early-strength cement.

THE CONCLUDING ARTICLE of five, analyzing the large retaining-wall tests carried out at M.I.T. some time ago, is presented in this issue. Dr. Terzaghi shows how the test results, using actual soil from the Fifteen-Mile Falls hydro development, conformed with the principles deduced from testing with special sand and fine-grained soils, as set forth in the early articles of the series. Application of the results to the actual design work will be set forth in a later article.

THE SEVERE WINTER just past has multiplied many-fold the instances of frost-heave difficulties on highways. A survey of causes and remedies, made by Henry Aaron of the U. S. Bureau of Public Roads, is therefore of timely and practical importance.

CONCLUDING the series of ten articles on "The Business of Contracting," J. L. Harrison writes on the management of construction labor, which on modern mechanized work is a problem of organization and balance between machine and manual operations as contrasted with the old idea of driving the men by bluff and brute strength.

Coming Articles:

VIBRATION as a method of compaction has in the last eighteen months come prominently into the concrete roadbuilding picture. As an outstanding development in the technique of concrete-pavement construction, a symposium of research and experience by the Bureau of Public Roads and several leading state highway departments has been prepared for publication next week. This will be the first published survey of general scope that has been made of a study that promises to play an epochal part in concrete roadbuilding.

Reprints of Code Digest

The article "How the Code Affects the Contractor—A Digest and Catechism," published in *Engineering News-Record*, April 5, 1934, is now available in four-page pamphlet form. Single copies, 5c. each, postpaid; quantities of 25 or more, 3c. each, postpaid.

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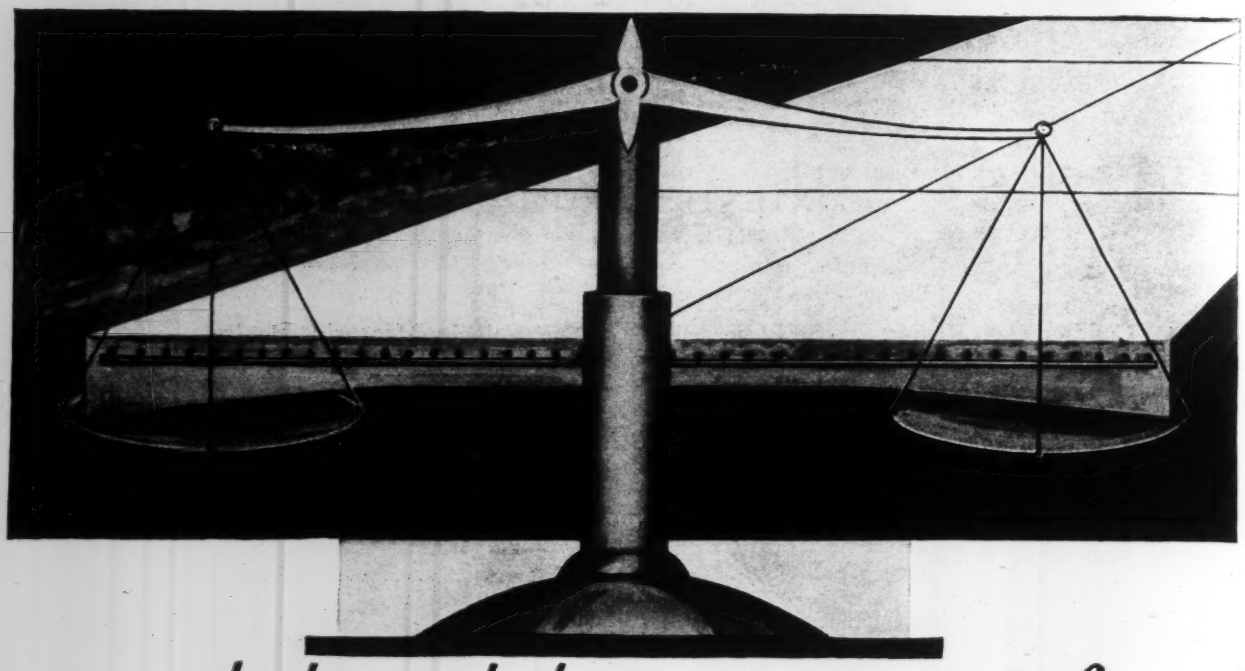

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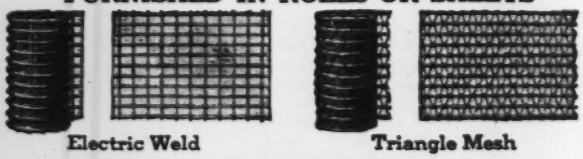

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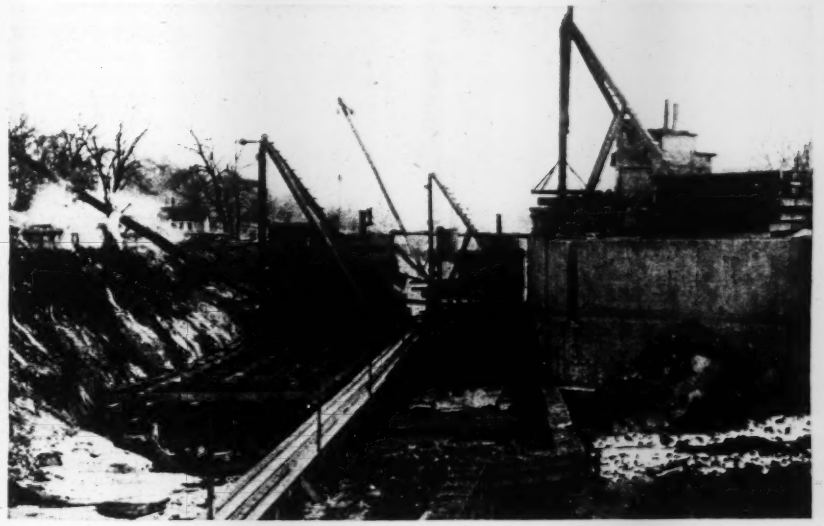


FIG. 1—THREE-DERRICK SET-UP for construction of Rapide Croche Lock on Fox River, Wis. The derricks and mixing plant move on rails. Boom on left derrick is reaching into heated-aggregate storage pile. Narrow-gage track in center carries cement and form cars.

Winter Concrete Construction on Rapide Croche Lock

Army engineers develop efficient heating system for cold-weather concrete placing on Lower Fox River in Wisconsin—Mixing plant and three stiff-leg derricks all travel on rails

REPLACEMENT of the 40-year-old lock at Rapide Croche, on the Lower Fox River near Kaukauna, Wis., necessitated that all work be done in winter seasons, to avoid interference with water traffic. At the close of the 1932 navigation season the Corps of Engineers, U. S. Army, started work on the new lock, contemplating a three-year construction program. The first season was devoted to excavation and the building of a retaining wall and the right lock wall. Despite the extreme cold weather prevailing during the past two winters, work has progressed beyond expectations and will probably be completed by the time navigation is resumed late this spring. Because of uncertainties of possible working time, it was decided to build the new locks by day labor. Interesting and efficient cold-weather plant and methods were developed.

The Rapide Croche lock installation is one of a series on the canalization of the Lower Fox River between Lake Winnebago and Green Bay, serving extensive paper-mill properties. The original lock and dam were about 40 years old. Several years ago the old timber dam was replaced by a new concrete structure about 1,500 ft. above the original lock and dam, a canal connecting the upper pool and the old lock. Slow settlement of the old lock, because of poor foundation, made the present installation necessary. The new lock is founded on rock, located near the new dam.

Though the lock is comparatively small, both its design and construction are of interest. The lock chamber, 171 ft. c. to c. of quoins and 36 ft. wide, provides for a 9-ft. maximum lift. All walls are gravity sections, about 30 ft. in height. A gravity-section retaining wall, about 250 ft. long, connects the left end of the dam with the right wall of the lock. About 8,000 cu.yd. of concrete is involved in the work.

Each of the lower gate leaves is 19 ft. $2\frac{3}{4}$ in. wide and 28 ft. $7\frac{7}{8}$ in. high, built up of nine 15-in. horizontal I-beams and two 15-in. ship channels, with a steel-plate facing, stiffened with a 1x3-in. strut strap. Quoin posts and miter posts are 15-in., 70-lb. I-beams. Independent operation of the four gates will be by hand. A racked spar fastened to the miter end of each gate will be extended or withdrawn by means of a bar-

FIG. 2—PORTABLE MIXING and batching plant add to flexibility of construction equipment set-up. Batcher moves on rails while steam mixer is shifted by the derrick.

operated ratchet mounted on a tripod. Tripod platforms are cantilevered over the outside of the lock walls.

Control of water will also be unusual. Intake valves will be located in a valve platform above the upper gates, controlled from the top of the walls. The outlet valves are of the butterfly type, built into the lower gates. These will be operated by a bar ratchet and pinion from the top of the gates.

Description of winter construction will be confined to the past season's work, as the previous year's operations on the retaining wall and right lock wall were similar, though of greater volume. Faced with the necessity of carrying on regardless of weather, the engineers equipped the job to meet the worst winter conditions. Two army boats, the dipper dredge "DePere" and the clamshell dredge "Winneconne" were anchored in the pool above the upper cofferdam, to provide steam for heating aggregates and concrete. Shop facilities of the dredges were used for equipment repair work.

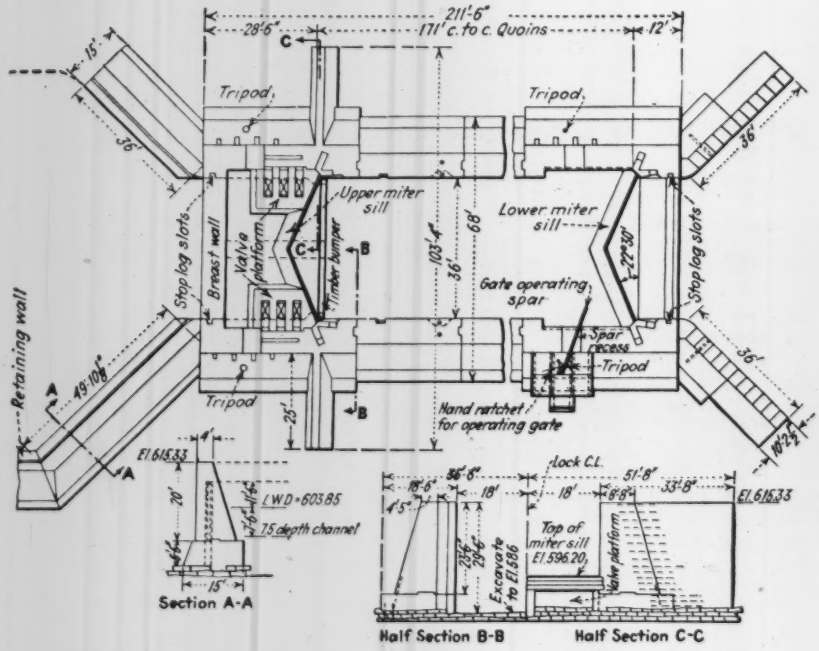
Closure of the lock site was effected by rockfilled timber cribs, backed with clay and sheeted for watertightness. Three traveling steam stiff-leg derricks comprise the backbone of the construction plant, one operating on top on the right bank and lock wall, and the other two working on the rock bottom and the lock floor. All operate on wide-gage tracks, those on the bottom being carried on timber cribs. The concrete-batching and mixing plant also operates as a unit on tracks cribbed up on the lock floor. A narrow-gage railroad on the lower level aids in handling heavy materials. The flexibility of this portable equipment is responsible to a great extent for the excellent progress made to date.

Excavation was fairly light, as the lock is being built in the old canal. Orange-peel and clamshell buckets, operated from the derrick, handled most of the excavation. Spoil material was piled on the banks for subsequent backfill. Additional backfill will be obtained from dredging of the canal bottom between the old and new locks.

Along the left bank was built a long, shallow sand and gravel storage bin about 40 ft. wide, and of timber. Before any material was received, a series of 1-in. perforated pipes was laid transversely across the bottom of the bin about 4 ft. apart. Along the outside of the bin was placed a steam header pipe, with a valve connection to each of the transverse pipes. The lower ends of the transverse pipes were fitted with valves for cumulation of the steam and for drainage. The header pipe was connected to the boilers of the two dredges for steam supply.

Practically all of each year's requirement of sand and gravel was received by barge before close of navigation, although about one-fourth of the total was trucked in later. All the material was stored in the long bin, piled directly on the transverse pipes within reach of one of the derricks in the hole. The pile averaged 15 ft. in height.

FIG. 3—PLAN AND WALL SECTIONS of Rapide Croche lock. The four gate leaves are operated by hand ratchets through spar connections as shown in the lower right-hand corner of the lock plan.



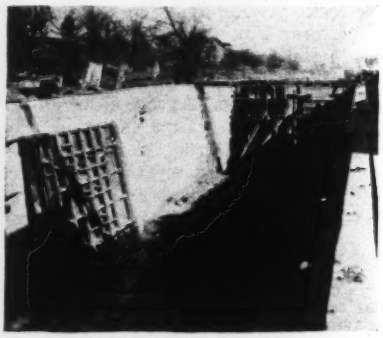
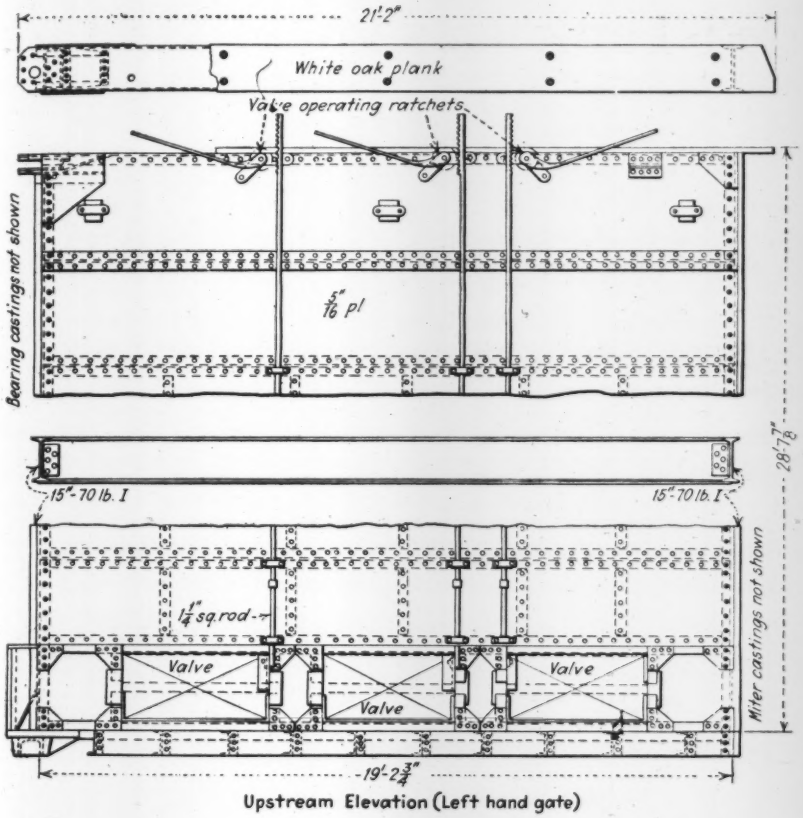
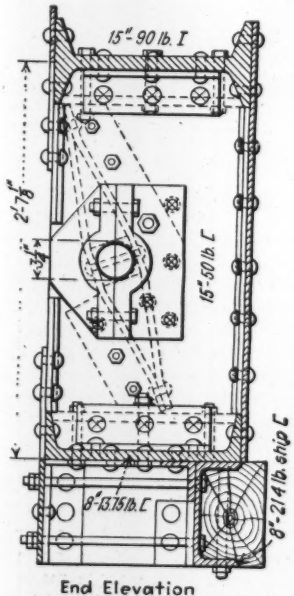


FIG. 4—DOWNSTREAM END of completed section of lock, with lower gates being erected in background. Heavy panel forms in foreground are typical of those used for wall construction.



Upstream Elevation (Left hand gate)



End Elevation
Assembly of Valve and Seal

FIG. 5—HAND-OPERATED VALVES are located in the lower gates. The four gate leaves are identical with the one shown here except that the inlet valves are in a valve platform above the upper gates instead of in the gate leaves.

With the coming of cold weather, parts of the stockpile to be used first were covered with canvas. From four to six days prior to start of placing concrete, depending on the weather, steam was admitted to transverse pipes under the sand pile to be used. About 48 hours prior to concreting, steam was admitted to the gravel storage. Regardless of air temperatures, the materials became evenly heated under protection of the canvas. On the coldest days the top of the stockpile registered temperatures of 150 deg.

Concrete work on the left lock wall started at the lower end, that on the right wall having been started at the dam and carried down to the lower end. One of the lower derricks equipped with a clamshell charged the portable batcher. Mixing water and materials in the hopper were heated by steam. Cement bags were chuted from a storage shed to a small car on the narrow-gage railroad for transportation to the mixer. The mixer was a 1-yd., steam-operated tilter, discharging directly into a 1-yd., controllable, bottom-dump bucket. This bucket was handled directly to the forms by either the second lower derrick or the one on top, or by both when a transfer was necessary.

Each form was completely incased in canvas and kept warm by steam piped to radiators and pipe coils from the dredges. Excellent concrete, testing above the required 3,000 lb., was produced. Heavy wooden panel forms were standard, built up for full height of wall of 3-in. facing, with heavy studs, all bolted together. Handling and setting of forms was by derrick.

Army engineers in charge report that, so far as equipment is concerned, concrete can be placed in any weather with this set-up. However, it has been possible to avoid work in temperatures below zero because of discomfiture to the workmen.

Erection of the steel gates in place is being carried on by day labor along with the other work. A large portable compressor furnishes air for the riveting hammers. Gates are being erected in half-open positions with the help of the top derrick.

Organization

Design and construction of the entire work are under the direction of the U. S. Engineer Office at Milwaukee. Col. H. M. Trippe is district engineer in charge. O. M. Skrukrud is resident engineer on the Fox River. J. E. Bambery is engineer in charge of construction. Plans and designs were prepared by Earl M. Nisen, senior engineer, and Albert R. Striegl, civil engineer.

FIG. 6—TARPAULIN-COVERED PILES of aggregates were heated with steam from perforated pipes laid on the bin bottom. Steam from two dredges was admitted to individual pipes as desired.

Defective Elevated-Tank Design Revealed by Earthquake

Principal points of vulnerability in collapsed South Gate, Calif., tank were at tower-rod connections—Design did not follow standard specifications in several particulars

By John Huber
South Gate, Calif.

OF THE HUNDREDS of elevated steel water tanks in the territory affected by the Southern California earthquake of March 10, 1933, only one, located in the city of South Gate and about 10 miles from the city of Long Beach, the center of the disturbance, crashed to complete destruction. Being a resident of South Gate, the writer had a first-hand opportunity of studying this collapsed tank. These studies lead to the conclusion that the failure was due to a lack of balance in the material used and also to the weakness of details and connections. These defects are made clear by comparing the tank design with the regulations of the National Board of Fire Underwriters or the specifications of the Associated Factory Mutual Fire Insurance Companies, which are identical. It is safe to state that at least 90 per cent of the tanks in the affected area were built in accordance with N.B.F.U. regulations, whose value was proved by the splendid way in which these structures stood the severe shocks.

Eyewitnesses to the collapse of the tank relate various versions, depending upon the location from which they viewed the failure. A man living within 100 ft. of the structure said that when he first felt the shock and ran outdoors he looked up at the tank, which at that time appeared to be about 20 deg. from the vertical; the columns moved and shook violently at the joints, and the tank collapsed during these phenomena. The momentum of the water burst the tank, and the water, spreading in streams, washed away his garage, caught him as he was running and tumbled him about. He was very close and his view would be that of one looking up. Another witness to the collapse stood about 600 ft. away from the structure. He stated that when first he noted the tank it was rotating horizontally in ever-increasing arcs; that the columns shook at the joints, followed by the collapse of the structure.

The view, Fig. 1, was taken the day following the collapse and shows the flattened condition of the tank. From the study of the wrecked structure it was clear that, in falling, two columns folded under the tank and were covered by the flattened plates, while the other two columns fell on top of the tank after it hit the ground. The tank was built by and according to the design of the Baker Iron Works, of Los Angeles, a firm no longer in business.

The drawing, Fig. 2, was made by the writer from data compiled from measurements of the wreckage and from available records. A complete redesign of the structure was made. In the following, only as much of this design analysis is given as will afford the reader a general understanding of the problem and will give support to the conclusions drawn.

FIG. 1—WRECKAGE of 150,000-gal. elevated tank on 100-ft. tower at South Gate, Calif., following earthquake of March 10, 1933.

The tank, of hemispherical-bottom type, stored a municipal supply and was of 150,000-gal. capacity, 26 ft. in diameter, with a cylindrical shell 29 ft. high. The height of the structure to the lowest opening was 100 ft.

In so far as the general design was concerned, the rods employed in the windbracing indicate that the tank was intended to withstand a 30 lb. per sq.ft. wind pressure, the usual practice to follow. Wind allowance, as usually specified, was taken advantage of in the design of the columns. The tank itself was the strongest part of the structure, being built of heavier material than called for under the N.B.F.U. regulations. Unit stresses in the joints of the shell and bottom were very much lower than could allowably have been used. Rivet shearing stresses were also conservative, as was bearing of rivets on plates. A check on the stresses in the upper rings also revealed that the stresses were well within the allowable working limits.

Columns were of the same section throughout—namely, two 15-in., 33.9-lb. channels with one 18×$\frac{7}{16}$-in. plate and lacing bars either $\frac{7}{16}$ or $\frac{3}{8}$ in. thick by $2\frac{1}{2}$ in. wide. The l/r of the columns was 75.4. Computations showed the unit stress in the columns to be 12,500 lb. per sq.in. The distance between rivet centers in the lacing bars was 21 in. Inasmuch as the Fire Underwriters' regulations require that the thickness of lacing bars be not less than 1/40 of the distance between rivet centers, the thickness of the bars should have been 21/40, or 0.525 in. Actually, as stated, they were $\frac{7}{16}$ and $\frac{3}{8}$ in. Also, the Fire Underwriters' regulations limit the maximum compression stress to 12,000 lb. per sq.in., although the A.I.S.C. specifications allow a unit stress of 13,671 lb. per sq.in. for the same l/r. Nevertheless, it would seem that the lacing bars were too thin for sound engineering practice.

The struts, on the other hand, showed a unit compression stress of 2,250 lb. per sq.in. as compared with an allowable 9,500 lb. The details shown on the drawing also indicate that the struts were well constructed. The same is true of the rods, which were made of $1\frac{1}{2}$-in. round bars not upset and were stressed to only 16,800 lb. per sq.in., as compared with an allowable 18,000 lb.

The balcony stiffening girder, being a vital element, was examined very carefully to see that the bending moment could be carried entirely by the flanges and the shear by the web as required by the Fire Underwriters' regulations. The analysis showed the girder to have been very stiff and, as indicated by the wreckage, the girder resisted the bending moment to the extent of burst-

ing the rivets fastening it to the shell. Some sections were bent more than others, but there was one section that appeared to be true to shape and free from twisting after the bursting of every rivet connecting it to the shell and bottom.

Causes of failure

On the accompanying drawing the ringed figures indicate the parts of the structure that showed failure. The calculations that follow reveal weaknesses in design that would account for the failure.

Among the principal elements that failed were the wing plates on the tower columns, to which the bracing rods were attached. In these the material tore out ahead of the clevis pins. The maximum load on the 1½-in. round bars in the upper panel was 18,750 lb. from wind and 3,000 lb. initial, or 21,750 lb. Inasmuch as the wing plates were ⅜ in. thick and the pin 2 in. in diameter, the bearing stress was

$$\frac{21{,}750}{2 \times 0.375} = 29{,}000 \text{ lb. per sq.in.}$$

The maximum allowable under the Fire Underwriters' regulations is 27,000 lb. per sq.in.

The distance from the edge of the punched hole to the edge of the wing plate was 1¼ in. The net section ahead of the pin, therefore, was 1.125×0.375, or 0.469 sq.in. The net area required by the Fire Underwriters' regulations is 62.5 per cent of the net area of the tension member, in this case $62.5 \times 1.294 = 0.809$ sq.in. Therefore, the edge distance should have been 0.809 / 0.375, or 2.16 in.

These figures indicate clearly that there was not enough material ahead of the pins to withstand the bearing pressure of the rods, also that there was not sufficient edge distance. As the earthquake shook the structure, the material ahead of the pin broke through, as shown in the drawing. This was no doubt the principal cause of the failure. The holes being punched instead of drilled and being $\frac{3}{16}$ in. larger than the pin may also have been a contributing factor to the failure.

After the tank fell, pins were scattered all over the area occupied by the structure. In some of the pins the cotters were missing, leading to the conclusion that during the earthquake the vibrations and stresses may have sheared the cotters, thus releasing the rods and causing the failure. Fire Underwriters' regulations require that pins either should be headed on one end and threaded on the other, or that both ends should be threaded and fitted with nuts, the threads to be burred over the nuts.

All of the rivets connecting the columns to the balcony girder were sheared off. The horizontal component due to live and dead loads was 45,150 lb. The connection consisted of eight ⅝-in. rivets. The allowable working stresses of the eight rivets at a shear value of 13,500 lb. per sq.in. would be $8 \times 0.307 \times 13{,}500 = 33{,}136$ lb. This is some 12,000 lb. short of the horizontal component of the load carried, explaining the failure of these connections.

All of the columns were twisted, bent or torn at the point near the top where the channels were cut to make room for the angles that connected the columns to the tank shell. The column plate was not carried far enough up on the angles; the six rivets which connected it to the angles were sheared off in all four columns.

From the above it may be concluded that the failure of the South Gate tank

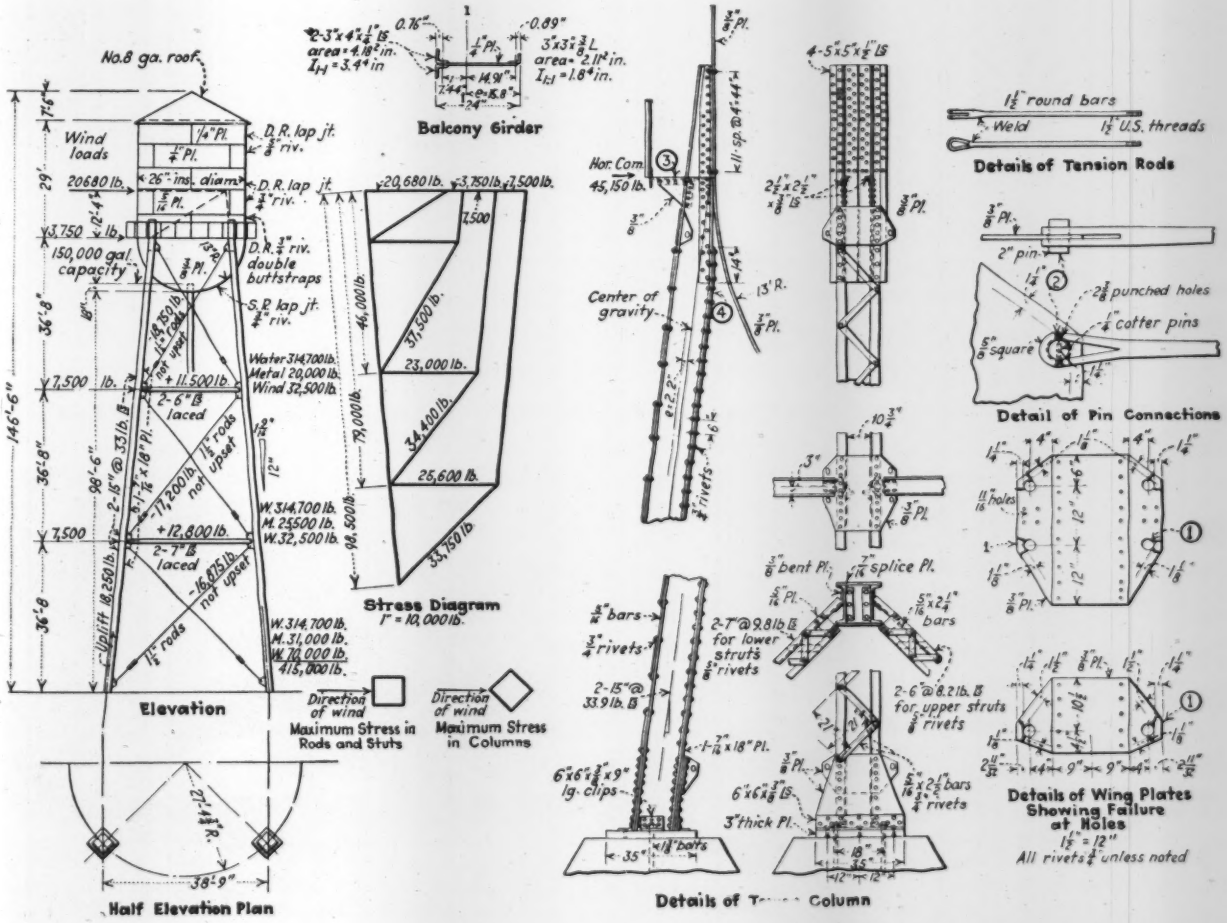


FIG. 2—DESIGN DETAILS of wrecked tank as drawn up by the author. Numbers in circles indicate points of failure. Tearing out of wing plates at tower-rod connections was probably prime cause of collapse.

was principally due to lack of balance in design. The extra material that was used in the first ring and bottom would more than have made up for the lack of material at points where it actually was needed.

Other elevated tanks

Nearly all tanks in the earthquake area had to be overhauled, most of the work consisting of tightening and renewing tower rods. Rods were affected differently, according to their make-up. On square bars with the threads welded to the bars, the shocks opened the forge welds at the scarfed edges; those that were not slit badly could be rewelded. Rods with upset ends stretched in the solid bar. This stretch was not uniform over the bar but occurred at several different points in lengths of about 12 in. The thickness of the bars was visibly reduced. Such rods in a tank at Huntington Park stretched 7 in. Some solid rods with ends not upset broke at the threads. The threads stretched before breaking, resembling the threads of lag screws.

The experience in this earthquake seems to indicate that rods with upset ends are superior to solid bars or rods with welded ends.

Tanks with riveted risers did not fare better than tanks with wrought-iron or cast-iron risers. Quite a number of tanks showed leaks and had to be calked, particularly at the connection of the tower legs to the tank.

New regulations

To assure greater safety for elevated tanks, the Board of Fire Underwriters of the Pacific has issued revised specifications applying to the states of California, Montana and Nevada and the territory of Alaska. The most important revision involves a lateral-force provision which is to supersede the wind provision in zones of known earthquake activity. In no case, however, may the size of any member, tierod, connection, strut or footing be less than that required by dead load plus wind load, or dead load plus live load alone. The lateral-force provision states that for tank towers 50 ft. or more in height located on the ground (and not on a building or extending through a building) details and members shall be designed to resist the stresses produced by horizontal shear equal to not less than the following percentages of the weight of the tank when full:

Height of Tower	Firm Ground	Filled Ground
75 ft. or more	8%	10%
50 to 75 ft.	12%	15%

Corrugated-Metal Arch Widens Concrete Bridge

Winter-time extension of existing arch a feature of road-widening project in Clinton County, Indiana

By Herbert Louck
County Surveyor and Road Supervisor, Clinton County, Ind

THE Jefferson-Mulberry Road in Clinton County, Ind., connects Mulberry, a community of 1,000, with Frankfort, the county seat. Last fall this road had a number of sharp curves, some extremely narrow fills and one narrow bridge located on a curve. Because of its dangerous condition and the amount of traffic that it carries, this road was one of the first singled out by our county authorities for improvement under the CWA.

All narrow fills were widened sufficiently to permit two cars to pass with a comfortable margin of safety. Additional right-of-way was secured so curves could be lengthened. It was also decided to widen the bridge, which consisted of a concrete arch with a span of 24 ft. and a rise of slightly more than 8 ft. At the time this decision was made it was already midwinter. Since there was no assurance that CWA labor would be available after Feb. 15, winter construction was necessary.

About a month previous, on another project in a different part of the county, a dangerous kink had been removed by extending an existing brick arch with a corrugated multi-plate arch. Experience on this small structure demonstrated the adaptability of multi-plate construction for bridge widening and also showed it to be so well suited to winter-time construction with unskilled labor that it was chosen to widen the concrete arch bridge on the Jefferson-Mulberry Road. (See *ENR*, Nov. 19, 1931, p. 805, for a description of multi-plate pipe.)

A check-up on the size of the existing structure showed that a metal arch could be secured that would approximate it closely in size. By making the extension 7½ ft. in length, a roadway with a clear width of almost 24 ft.—the county standard for all new structures—would be provided. Placing the extension on the downstream side of the arch and on the inside of the curve also made it possible to improve the road alignment.

First, footings were poured. To insure proper setting of the concrete, both the water and aggregate were heated. Once in place, the footings were easily protected against freezing because they extended only a few inches above ground level.

After the footings had been given ample time to cure, the arch was erected. To provide for a closer fit with the old arch and to conform with the alignment of the road, the new arch was placed on a slight skew by offsetting the plates.

Following the erection of the metal arch, a brief period of unseasonably warm weather gave an excellent opportunity for pouring the headwalls. Due to lack of time and the season of the year, however, the construction of the wingwalls was left for some future time, and provisions were made for joining these to the headwalls already in place.

The improvement of this road was made under the supervision of the writer. Bill Michael, assistant supervisor, was in charge of the extension of the arch.

CONCRETE ARCH widened in winter by adding a ring of corrugated metal and building new headwall.

Determining Bearing Power in Earth Foundations

Tests, theory and interpretation of bearing tests conducted for foundations of an elevated water tank

By H. J. Summers
Assistant City Engineer, Torrance, Calif.

THE PURPOSE of this article is to describe a test, interpret the results in terms of the theory herein discussed, and to suggest, or perhaps speculate on, the possibilities of a new method of laboratory testing using core samples of soil, thus eliminating the time and expense required to construct and operate the loading platform tests.

Description of test

A test was conducted at Torrance by the writer in October, 1933, at the site of a proposed elevated steel water tank. The soil was fine sand having a high percentage of yellow clay, but no loam, with a moisture content of 6 per cent by weight. A pit was dug 6 ft. deep, that being approximately the depth of the proposed footings. A 12x12 in. post was set in the pit. On this was constructed a platform 8 ft. square (Fig. 1). Bracing allowed free vertical motion of the platform, but no lateral or rotary motion. On the platform was placed a 625-gal. water tank. The tank, platform and post weighed 925 lb., this being the initial loading. The tank was too small to give sufficient loading on a square-foot test, and a ton of pig iron was used to increase the load. This was weighed as it was placed on the platform.

A drawing board was arranged with a pantagraph to multiply the vertical motion of the post by 5 and connected directly to the post under the platform. Ordinary profile paper was used on the board, as the smaller divisions are 1/20 in., representing .01 in. of actual settlement. It was found that .001 in. could be approximated very well.

By auto jacks the post could be raised from the ground and moved over after each test; each time the platform and post were leveled and plumbed. The weight of each load increment was determined by measuring the volume of water in cubic feet and multiplying by 62½; the water depth was measured in four different places and averaged.

In addition to the 1-sq.ft. bearing area, tests were made using 1 sq.ft. round, ¼ sq.ft. square and ¼ sq.ft. round plates. These were wood plates 4 in. thick—merely set under the 12x12-in. post. Load increments of 500 lb. were tried until it was shown that the curves were quite regular and smooth, when 1,000- or 1,500-lb. increments were used.

The actual field readings will be omitted here, the curves being given to show results. Some difficulty was experienced with water getting into the pit. For this reason all of the curves are not reproduced, but only those that are thought to be comparable. In the lower range of loadings the curves were exactly a straight line. Therefore, as the initial load of 925 lb. could not very well be eliminated, the slopes of the curves were extended to 0 load, and thus the settlement for the initial load was determined. The curves are all corrected to pass through the 0 point.

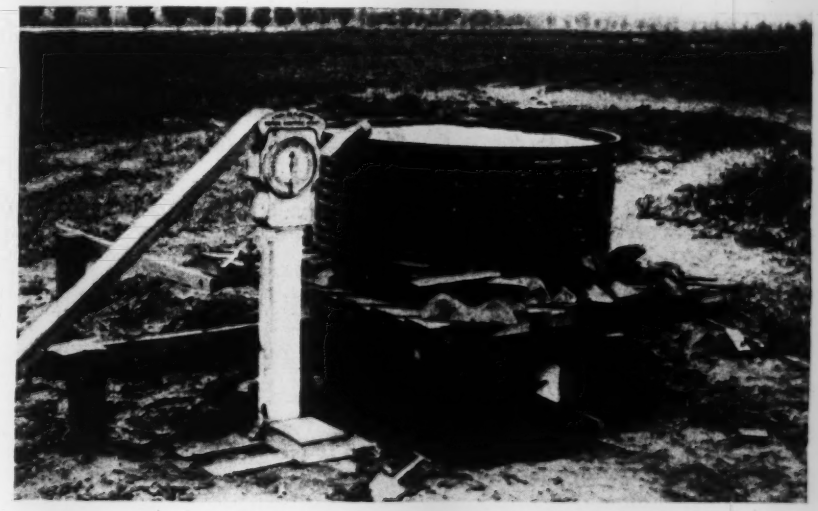


FIG. 1—APPARATUS for making soil bearing-power tests for water-tower foundations at Torrance, Calif.

Presentation of theory

A fundamental fact in soil mechanics, as disclosed by Paaswell, Terzaghi and others, is that a strained soil offers resistance to the stress in two ways: by its direct compressive strength under the applied load and by its shearing strength around the perimeter of the bearing area. Research has also shown that soil under load has two different characteristics dependent upon the load. With light load the characteristic is that of a plastic, somewhat elastic medium. With heavy load it approaches that of a fluid medium. The transition between the two, as the load is increased, is more or less sudden or gradual, depending upon a number of rather obscure factors. The equation of loading may then be established as follows:

$$W = Ap = Pm + An,$$

Dividing by A,

$$p = \frac{P}{A} m + n \qquad (1)$$

Let $\frac{P}{A} = x$. Substituting x and dividing by Δ,

$$\frac{p}{\Delta} = \frac{m}{\Delta} x + \frac{n}{\Delta} \qquad (2)$$

Where: W = total load on footing.
A = area of footing.
P = perimeter of footing.
m = value of perimeter shear or soil shearing strength.
n = soil compressive strength.
p = load per unit area.
Δ = unit settlement.

Tests conducted at the University of Michigan in 1929 with various sizes of bearing areas entirely checked Eq. (1) for the lighter zones of loading.

Eq. (2) may be interpreted to mean that, for unit settlement, the load is counteracted by the strength of the soil in compression (at that settlement) plus the shearing strength of the soil (at that settlement), the latter quantity being proportional to the perimeter-area ratio.

In the loading of soils the quantities $\frac{m}{\Delta}$ and $\frac{n}{\Delta}$ in Eq. (2) are parameters remaining more or less constant in the region of lighter loadings, but varying considerably in the region of heavier loadings. As soil failure takes place and under heavy loads the behavior of the two parameters is sometimes very erratic and quite different for different soils and soil conditions. This article, therefore, is not primarily concerned with the regions beyond soil failure, but rather with the point at which failure begins to take place and with the regions of lighter, hence safe, loads.

To determine, then, the two unknown

parameters $\frac{m}{\Delta}$ and $\frac{n}{\Delta}$ in Eq. (2), it is necessary to make two or more tests with different values of x, the perimeter-area ratio. To do this, get load-settlement data with different sizes and shapes of the bearing area. Substitute these data, obtaining a set of simultaneous equations (2). Then, by the method of least squares, find the most probable values of $\frac{m}{\Delta}$ and $\frac{n}{\Delta}$. These will be substitued back in (2), the resulting equation expressing the relationships between load, settlement and perimeter-area ratio for that particular soil.

As the perimeter-area ratio (the quantity x) is large for small areas and small for large areas, it will be seen that the test will show a larger load per unit settlement than can be expected in the finished foundation, as the test area is small.

The shape of the load-settlement curve is similar to the stress-strain curve in elastic materials. It starts out with settlement proportional to load, or nearly so, and finally curves over. At this point and beyond it the element of time between increments of load becomes important—i.e., after the load is applied, settlement continues progressively for some time before the footing finally comes to rest; the soil is in an unstable condition. Before this point is reached on the curve, the load is applied, settlement taking place immediately and quickly ceasing. In conducting the test, the time element should be eliminated when working in the zone of progressive settlement by allowing the same time to elapse between successive load increments. This may be 15 or 30 min. A sample time-settlement curve is given in Fig. 2, this being in the first test pit with the ¼-sq.ft. round plate at 20,400 lb. per sq.ft. load.

Tests on a soil should be conducted up to sufficient loads to determine the characteristic shape of the load-settlement curve and the region where the progressive settlement begins. As with other materials, the safe load would be the load corresponding with the beginning of the progressive settlement divided by the factor of safety. Failure to determine this point would obviously lead to erroneous conclusions.

Interpretation of results

Referring to the curves for test pit No. 1 (Fig. 3) and to the equation

$$\frac{p}{\Delta} = \frac{m}{\Delta}x + \frac{n}{\Delta}$$

it will be seen that the slope $\frac{p}{\Delta}$ of the curve for $x = 3.545$ (1 sq.ft. round) is in the neighborhood of 42,900 (from the data, 42,920). Similarly, $\frac{p}{\Delta}$ when $x = 8$ is 54,400. Substituting in the equation,

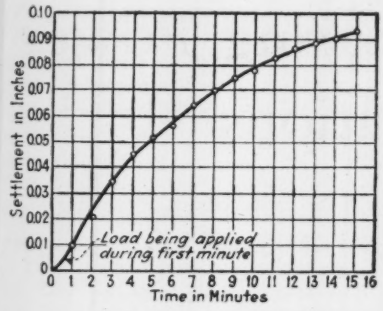


FIG. 2 — TIME-SETTLEMENT CURVE in the range of heavy loading, showing progressive settlement.

we have a set of simultaneous equations:

$$\begin{cases} 54{,}400 = 8\cdot\frac{m}{\Delta} + \frac{n}{\Delta} \\ 42{,}920 = 3.545\cdot\frac{m}{\Delta} + \frac{n}{\Delta} \end{cases}$$

So that

$$\frac{m}{\Delta} = 2{,}580, \text{ and } \frac{n}{\Delta} = 33{,}780$$

These values of the parameters will give the desired equation:

$$\frac{p}{\Delta} = 2{,}580x + 33{,}780.$$

If the proposed foundation is to be 12 ft. square, $x = \frac{P}{A} = 0.333$ and $\frac{p}{\Delta} = 34{,}639$. This is the slope of the straight part of the curve for the projected foundation, or, putting it another way, it expresses the expected value of pounds per square foot load per inch settlement.

Reference to the curves, Fig. 3, shows that the point where failure begins is approximately 0.22 in. Behavior of soils seems to indicate that failure will begin in every case at this settlement rather than at the load corresponding to it. This is a safe working basis and means that 0.22 in. settlement is the safe limit in this case. With a factor of safety of 2, this fixes the allowable settlement at 0.11 in. and the allowable load thus:

$$\frac{p}{\Delta} = 34{,}639, \text{ when } \Delta = 0.11 \text{ in.},$$

$$p = 3{,}810, \text{ allowable load.}$$

Borings under each of the seven proposed foundation piers showed six conditions of soil to be practically identical, while the seventh showed a different proportion of sand to clay. A second pit was dug and a second test made. Three curves are shown for this test. As the curves are quite consistent in their slopes, the mean values were taken for $\frac{m}{\Delta}$ and $\frac{n}{\Delta}$, found by solving a set of three simultaneous equations. The resulting equation is

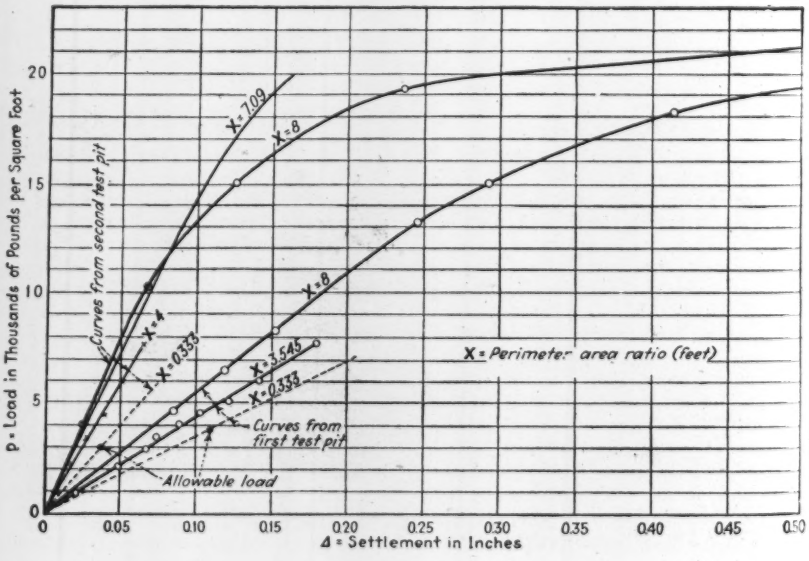
$$\frac{p}{\Delta} = 10{,}540x + 74{,}920.$$

This would seem to indicate a soil with high bearing capacity as compared with the first test, but strangely, on the basis of the foregoing analysis, it is not. This is because the settlement at which failure begins is less, being in this case in the region of 0.075 in. Substituting $x = 0.333$, the equation becomes

$$\frac{p}{\Delta} = 78{,}440. \quad \text{At } \Delta = .0375 \text{ in.}$$

$$p = 2{,}941, \text{ allowable load.}$$

FIG. 3—LOAD-SETTLEMENT CURVES from two test pits for soil-bearing tests at Torrance, Calif.



One thing should be pointed out. The factor of safety is in itself a somewhat flexible quantity. That is, in such a structure as an elevated water tank the loads computed to be transmitted to the foundations carry a certain factor of safety such as wind loads never to be realized, earthquake stress which is enigmatical and certain allowances here and there. In addition, it is advisable to be governed not entirely by the factor of safety, as the element of settlement must be considered. Uneven settlement may be less safe at times than a small factor of safety. In the case at hand, settlement for six piers is expected at .11 in. and for the seventh at .0375 in. This latter pier is located from the center transversely across the line of the prevailing winds and not in the direction of storm winds. Therefore it is felt to be not unreasonable to allow the same loading for this pier as for the others, viz., 3,800 lb. per sq.ft. The factor of safety in this case is 1.55.

It must also be borne in mind that these tests were conducted in open pits, there being no constraining influence of the sides of the pits. In an actual foundation structure or with a test in a pit the size of the bearing area, the early failure in some soils would be inhibited by such a constraint.

It was noted that in the second test, in one case of soil failure, there was a cracking and upward movement of the material adjacent to the bearing area.

It seems to be entirely within the realm of possibility that tests could be run in the laboratory on core samples of the soil under consideration to determine the values of $\frac{m}{\Delta}$ and $\frac{n}{\Delta}$. Difficulties would probably be:

1. Obtaining samples which would represent the true condition of the site —namely, the same moisture content and same degree of compaction.
2. Properly constraining the sample so that it would not be loose.
3. Devising means for measuring an equivalent of shear and compression.

Concrete Paving Successful in Sub-Zero Weather

Ohio highway paved with concrete in temperatures ranging from freezing to below zero and without stop because of the cold

By H. P. Chapman
Chief Engineer, Ohio Department of Highways, Columbus, Ohio

A DESIRE to further the President's re-employment program by carrying on highway construction during the winter months has led to adoption of practices unusual in the Ohio department of highways. Uncertain climatic conditions in Ohio had built up a precedent for the belief that concrete paving work should be abandoned at the first sign of freezing weather and not resumed until the next spring. In an effort to avoid laying off large numbers of men, numerous methods were tried with a view to extending this period, one of the most important of which was the use of high-early-strength cement. This material was specified on a number of paving projects, one of which was on the Hamilton-Eaton road in southwestern Ohio.

This highway was in Butler County on the east bank of the Miami River, and is one of two sections of a new route provided to handle through-traffic between Hamilton and Williamsdale. The paving consists of 6,040 lin. ft. of 9:7:9-in. concrete slab 30 ft. wide and 972 lin. ft. of the same type 41 ft. wide. The contractor was Crist & Beatty, of Hamilton, Ohio, whose bid price was $75,128. The contract was awarded on Nov. 22; operations got under way three days later, and the first concrete was poured Dec. 5, and the last Dec. 30.

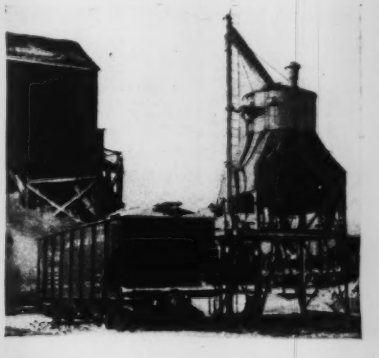
Equipment and methods

The usual cold-weather precautions were taken on this job. Finished grading operations were confined to within 500 ft. of the mixer, and the subgrade was protected from the weather with a covering of burlap and straw. Upon removing the covering any frosted subgrade was thawed out with hot water and steam from a water line connected to a 20-hp. upright steam boiler, which also furnished the heated mixing water for the concrete. This boiler was mounted on wood skids and was drawn along by the mixer. Forms were also preheated with steam and hot water preparatory to placing the concrete slab.

Freezing temperatures were encountered almost from the start; later the thermometer dropped below zero several nights, but concrete was placed continuously during the progress of the work, with the exception of one or two days when the subgrade was not ready for placing.

The aggregates were heated by steam pipes at the batching plant, and their temperature, together with that of the heated mixing water, resulted in the concrete being placed on the grade at

BULK CEMENT and heated aggregates for the paving mixer originated at this plant.



PAVING MIXER operated outside the forms pulled a steam boiler, which supplied hot mixing water and steam for heating forms and removing frost.

HAULING heated aggregate batches (above) and laying 70-deg. concrete during snowstorm (right).

an average of 70 deg. After finishing, the concrete was covered with wet burlap and straw to a depth of 10 in. This covering was maintained until the required beam strength (600 lb.) was obtained for opening the pavement to traffic.

Bulk cement was used and was delivered from the contractor's 80-bbl.-capacity storage hopper and weighing batcher set upon a siding at the commercial plant supplying the batched aggregates for the work. This plant was located about 2 miles from the center of the job. The cement was transported to the mixer in separate containers mounted on the batch trucks.

The paving slabs were finished with a gas-electric, double-screened finishing machine, which also supplied current for the lights used for finishing after dark. The mixer and upright boiler for heating the mixing water, and also the batch trucks feeding the mixer, were operated outside the forms, thus permitting the concrete to be placed on a wholly undisturbed grade.

This paving being on a new route, there were no detour problems to contend with. During construction, public traffic continued to use the parallel highway on the opposite bank of the river. A big item of the work, however, was the repair of a number of bad washouts in the high grade, caused by an abnormal rise of the adjacent river. These washouts affected about 1,200 lin. ft. of the grade for half its width. To repair the damage, about 37,000 cu.yd. of material was required, of which 8,000 cu.yd. was slag that had to be trucked about 1½ miles. The slag was used as a base for the new fill, and it was placed to a depth of from 8 to 10 ft. on the river bottom. A considerable quantity of timber and brush was used in building up and protecting the fill. A portable lighting plant permitted this work to be continued through part of the night.

About 360 men were employed on the job. These men worked in two five-hour shifts, except for the repairs to the grade, where three five-hour shifts were employed.

Results successful

Evidence that the concrete suffered no damage from the low temperatures is to be found, first, in the morning thermometer readings under the burlap and straw covering following cold nights, and, second, in the uniformly excellent results from strength tests of the concrete.

The 6x6x42-in. test beams were made from each day's pour and cured and protected with the pavement. Three breaks were made from each beam, using job apparatus. Similarly, two 6x12-in. cylinders were made from each day's pour and cured and protected with the pavement for approximately 40 hours, when they were sent to the state highway testing laboratory at Columbus for compression tests. Occasional cores were also taken for test.

The results of tests on 36 beams made to Dec. 23 revealed that the required flexural strength (modulus of rupture) of 600 lb. for opening pavement to traffic was obtained in a number of instances in 48 hours, the average being 72 hours. Cores cut from the pavement at the early periods averaged as follow

Age	Lb. per Sq.In.	No. of Te
24 hours	1,886	4
48 hours	2,341	4
72 hours	3,332	5
7 days	4,382	6
14 days	5,537	5

A particularly interesting test ser is that of the concrete placed Dec. The first night the temperature dropp to 7 deg. below zero. The next mor ing surface readings taken under t covering indicated a concrete temper ture of 44 deg. The air temperatu at that time was 6 deg. above zer Test cores 24 hours old showed a co pressive strength of 1,509 lb. On t third night the air temperature dropp to 5 deg. below zero. Again cores were cut and tested; this time the strength was 2,610 lb.

Welded Battledeck Floor Acts as a Unit Under Load

Tests made at the Bureau of Standards in cooperation with the American Institute of Steel Construction on a battledeck floor panel having a span of 18 ft. indicate that such a floor acts as a unit under load. The floor was made up of 4-in., 7.7-lb. I-beams spaced 2 ft. apart carrying ¼-in. plates 2 ft. wide welded together and to the I-beam along the center of the upper flange. Strain-gage and deflection readings were obtaind for live loads up to 200 lb. per sq.ft., after which the load was increased to 420 lb. per sq.ft., with no indication of collapse. There was no visible proof that the welds either ruptured or deformed permanently. It is concluded for ordinary spans that, provided the spacing of the beams does not exceed 100 times the thickness of the plate, the entire width of the plate may be assumed effective when designing such a floor. (See March number of the *Journal of Research.*)

LARGE RETAINING WALL TESTS

A series of five papers reporting fundamental results

V—Pressure of Glacial Till

Results of final tests on actual soil from Fifteen-Mile Falls Dam indicate close conformity to the principles deduced in preparatory studies of sand and fine-grained soils

By Karl Terzaghi
Professor, Technische Hochschule, Vienna, Austria

FOLLOWING the wall-pressure tests with dry and saturated sand, tests were conducted with a backfill of a natural modified till, or glacial drift. The sand tests, described in the first and second article of this series, had already led to a broader knowledge of the effect of submergence and of wall movements on lateral pressure. Without this empirical knowledge, no interpretation of the results of the tests with modified till could have been attempted.

The testing material

The modified till used in the tests was a glacial material of mixed grain size. Since it was formed by erosion and subsequent redeposition of a boulder clay, it differed from the boulder clays of the same locality merely by somewhat greater effective size and a smaller uniformity coefficient. A sample of 60 cu.yd. of this material was excavated at El. + 580 ft. near the center line of the Fifteen-Mile Falls Dam across the Connecticut River, on the New Hampshire side of the river.

Fig. 1 shows a typical grain-size curve for this material. The effective size was of the order of 0.15 mm., the uniformity coefficient about 20, and the coefficient of permeability variable between 0.01 and 0.10 $\times$ 10^{-4} cm./sec. Due to the high average specific gravity of the grains (2.8) and its mixed-grained character, the average weight of the saturated, compacted till (volume of voids 28 per cent) was as high as 143.5 lb. per cu.ft.

Outline of test procedure

Before the bin was filled, its bottom and rear wall were lined with a filter of Plum Island sand (Fig. 2). Later the objection was raised that the presence of a layer of foreign material at the rear wall of the bin might affect the test result. Therefore, while the test was going on the rear part of the filter together with the adjoining part of the fill was excavated and replaced by till. This operation had no effect whatever on the results. It did not even change the readings of the pressure dials at the retaining wall beyond the normal range of variation.

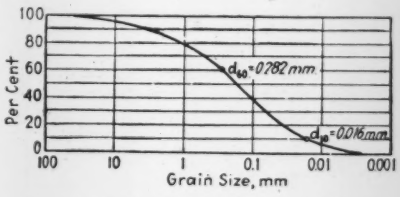


FIG. 1—GRAIN-SIZE curve of modified till selected from site of Fifteen-Mile Falls Dam.

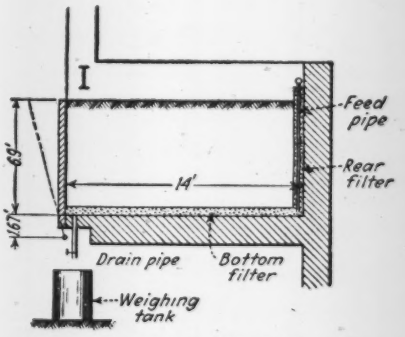


FIG. 2—ARRANGEMENT of bin for test of modified till from Fifteen-Mile Falls Dam.

The till was placed in the bin in 6-in. layers and compacted with concrete tampers. In its initial state the lower part had a moisture content of 8 per cent and the upper part of 11 per cent (per cent of dry weight); however, after the first submergence, this figure never dropped below an average of 12.6 per cent, which indicated that even in a drained state the fill was practically saturated, with an average unit weight of 142.9 lb. per cu.ft.

The test lasted from June 22 to Aug. 3, 1929, a period of 42 days. From its original position the wall was allowed to yield by increments, tilting around a horizontal axis located in the plane of contact between wall and fill (Fig. 2). The test comprised the following successive operations:

a—After the bin was filled, the saturation line (standpipe level) was allowed to rise to the surface of the fill, the wall remaining in its original position.

b—The fill was drained without moving the wall.

c—The wall was moved out an average distance of 0.15 mm. while the fill was in a drained state.

d—The fill was submerged and drained again.

e—The wall was moved out to an average distance of 0.45 mm. while the fill was in a drained state.

f—The fill was submerged.

g—The wall was moved out a distance of 0.85 mm. while the fill was submerged. Then the fill was drained.

h—The wall was moved out to a distance of 1.3 mm. while the fill was drained.

i—The wall was moved out to a distance of 4.2 mm. while the fill was in a drained state.

j—The plane of saturation (standpipe level) was raised and lowered through the full depth of the fill at stationary position of the wall four times at about two-day intervals, and the time change of the earth pressure was observed.

During the tests regular readings were made of the deformations of the machine, room temperatures and rates of evaporation, but none of these items was found to have an influence on the pressure phenomena sufficiently marked to be worth recording.

Results of the tests

The results of the tests are graphically represented in Figs. 3 and 4. In view of the facts presented in Article III we were entitled to make as clean-cut a separation between the pressure of the water and that of the solid fraction of the fine-grained backfill as was done when representing the results of the test with coarse sand in Article II. Since drainage produced little change in the water content of the fill, very marked capillary phenomena were to be expected. A brief description of the influence of capillary forces on the lateral pressure was presented in the conclusions of Article IV. However, in order to bring the results of this influence clearly into evidence, the observations on drained backfill were plotted *as if* capillary force did not exist. Hence the

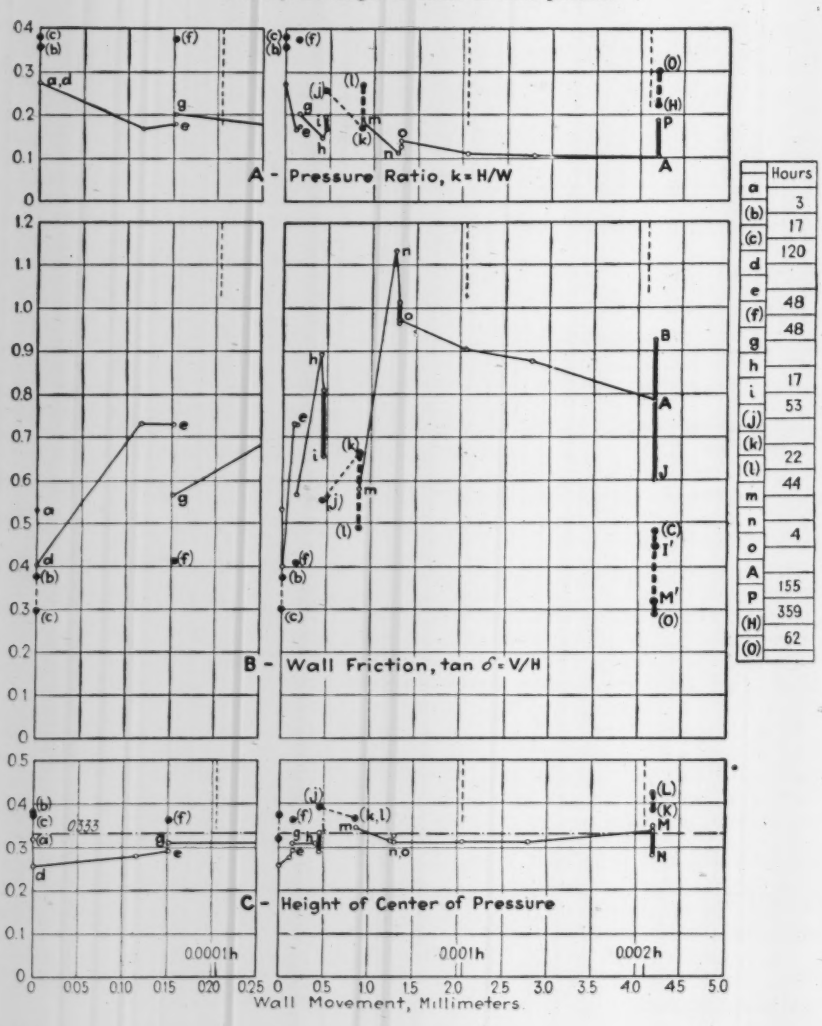
method of computing hydrostatic pressure ratio k, coefficient of wall friction tan δ, and height c of the center of pressure, was exactly identical with that described in Article II for the coarse sand submerged.

Diagrams *A* to *C* in Fig. 3 represent the relation between the average distance through which the wall was allowed to yield (abscissas) and the hydrostatic pressure ratio k (Fig. 3 *A*), the coefficient of wall friction tan δ (Fig. 3 *B*), and the elevation c of the point of application of the resultant earth pressure (Fig. 3 *C*). In these diagrams the points corresponding to the submerged state are connected by dotted lines, and the corresponding letters are shown in brackets. The transition between drainage and saturation is marked by gaps between vertical dotted and solid lines. The time intervals which elapsed between the successive stages of the test are shown in a table on the right side of the diagrams.

The last phase of the tests, marked in Fig. 3 by capital letters, consisted of repeated submergence and drainage at stationary position of the wall. The ordinates of the ends of the vertical lines which represent this state in Fig. 3 give the extreme values observed during the operations. The vertical solid line shows the range of scatter of the values for the drained, and the dotted line for the submerged state.

Fig. 4 represents those tests which in Fig. 3 are marked with capital letters, plotted on a time scale. In the diagram at the top of Fig. 4, abscissas represent the time which passed after the wall arrived at an average distance of 4.2 mm. from its original position, and ordinates the position of the water level in the standpipes which were connected with the bin. The ordinates in the diagrams *A*, *B* and *C* of Fig. 4 show the hydrostatic pressure ratio, the coefficient of wall friction and the height of the center of pressure. Finally, at the bottom is plotted the vertical movement of a point of the surface of the backfill 4.5 ft. back of the wall. The data contained in this diagram were also plotted *as if* no negative hydrostatic pressures existed in the drained backfill.

FIG. 3—RELATION of wall yield, hydrostatic-pressure ratio, coefficient of wall friction, and height of center of earth pressure.


A - Pressure Ratio, k = H/W
B - Wall Friction, tan δ = V/H
C - Height of Center of Pressure
0.0001h
0.001h
0.002h
Wall Movement, Millimeters

	Hours
a	
(b)	3
(c)	17
d	120
e	
(f)	48
g	48
h	
i	17
(j)	53
(k)	
(l)	22
m	44
n	
o	4
A	
p	155
(H)	359
(O)	62

Analysis of test results

Comparing the test results shown in Fig. 3 with those described in Article II for submerged sands, the following differences will be noticed:

For compacted dry sand the hydrostatic pressure ratio at the outset of the test was 0.58, the coefficient of wall friction 0.08, and the height of the center of pressure 0.4 h. In striking contrast to these findings the till showed at the outset, and prior to the first saturation (Fig. 3), a hydrostatic pressure ratio of 0.28, coefficient of wall friction as high as 0.53, and center of pressure somewhat below the middle third, at 0.32 h.

Another conspicuous difference appeared in the effect of submerging the drained backfill. In the tests with sand, after the wall was moved out to a distance of 4.2 mm. = 0.002 h, water was admitted into the fill, and the hydrostatic pressure ratio went up from 0.160 to 0.163 (increase of 2 per cent), the wall friction decreased by 9 per cent, from 0.476 to 0.435, and the center of pressure rose from 0.392 to 0.436 h. Hence the effect of saturation on the conditions of stress and strain which existed in the backfill was in these states very small. For this reason it was surprising to find that saturation of the till after the wall had been moved out to the same distance (Fig. 4, points B-C) caused very large changes, as follows: k went up from 0.11 to 0.252 (increase of 129 per cent), tan δ decreased by 50 per cent, from 0.927 to 0.463, and the center of pressure rose from 0.337 h to 0.385 h. At the same position of the wall repeated saturation and drainage caused the center of pressure to move between the extreme positions of 0.26 h and 0.42 h.

These differences between the pressure effects of sand and till are precisely what they should be according to the conclusions of Article IV. As a matter of fact, when the tests were made these differences came as a surprise, and the theory which explains them was worked out afterwards. Since the depth of the backfill was considerably greater than the capillary height for a column of dry till, the drained backfill was in a state of partial capillary saturation, with an average unit weight of 142.9 lb. per cu.ft. (water content of 12.6 per cent) against 143.5 for the state of complete saturation (water content 13.9 per cent). Hence in order to obtain the real values of k and tan δ for the drained state, the same separation must be made between the forces exerted by the water and by the solid fraction as was made from the beginning for the submerged state.

Capillary Forces—The computation of the negative hydrostatic pressure (capillary suction) is made by means

of the equations derived in Article IV. Two extreme cases can exist, viz:

1. An open fissure is formed between the backfill and the wall through the full depth of the suction zone. For this case we obtained:

Capillary suction, $p =$

$$\left(1 - \sqrt{\frac{H_4}{m\,k\,W_3}}\right) \frac{m\,k\,h\,s_3}{1 - m\,k} \qquad (1)$$

Depth of suction zone, $h - h' =$

$$h\left(1 - \sqrt{\frac{H_4}{m\,k\,W_3}}\right) \qquad (2)$$

Coefficient of wall friction, $\tan \delta =$

$$\frac{V}{H_4 + b\,p\,h'} \qquad (3)$$

2. The backfill adheres to the wall over the full depth of the fill, and no tension cracks are formed in the fill:

$$p = \frac{m\,k\,W_3 - H_4}{b\,h\,(1 - m\,k)} \qquad (4)$$

$$h - h' = \frac{p\,(1 - m\,k)}{m\,k\,s_3} \qquad (5)$$

$$\tan \delta = \frac{V}{m\,k\,W_3 + b\,h\,p} \qquad (6)$$

The coefficient m in these formulas expresses the increase of the hydrostatic pressure ratio k of the solid fraction of the backfill due to time effect and to the rearrangement of the grains produced by the processes of submergence or drainage at a stationary position of the wall. As admission or withdrawal of water was always performed after a long intermission while the wall remained stationary, we may for a rough estimate assume that $m = 1$. This assumption involves the possibility of an error of 15 per cent, or 20 per cent in the value of k.

Another source of uncertainty resides in the arbitrary character of our assumptions concerning the depth to which the open fissure between the fill and the wall extends. Both open fissures between the wall and the fill and open tension cracks within the fill were observed, but it was never possible to measure their depth except in one case of a tension crack which appeared about 3 ft. back of the wall after the wall arrived in its extreme position. Through a window in the wall of the bins it could be seen to extend almost vertically downward to a depth of about 1.84 ft. below the surface. For the first transition *B C* (Fig. 4) from the drained into the submerged state, which was performed at this position of the wall, we obtained by means of Eq. (1) to (3) the following values: $p = 125.5$ lb., $h - h' = 2.60$ ft., and $\tan \delta = 0.368$. Hence the depth of the fissure was only 70 per cent of the depth of the zone of suction, and the corresponding state was very likely intermediate between what was previously called state (a) and (b).

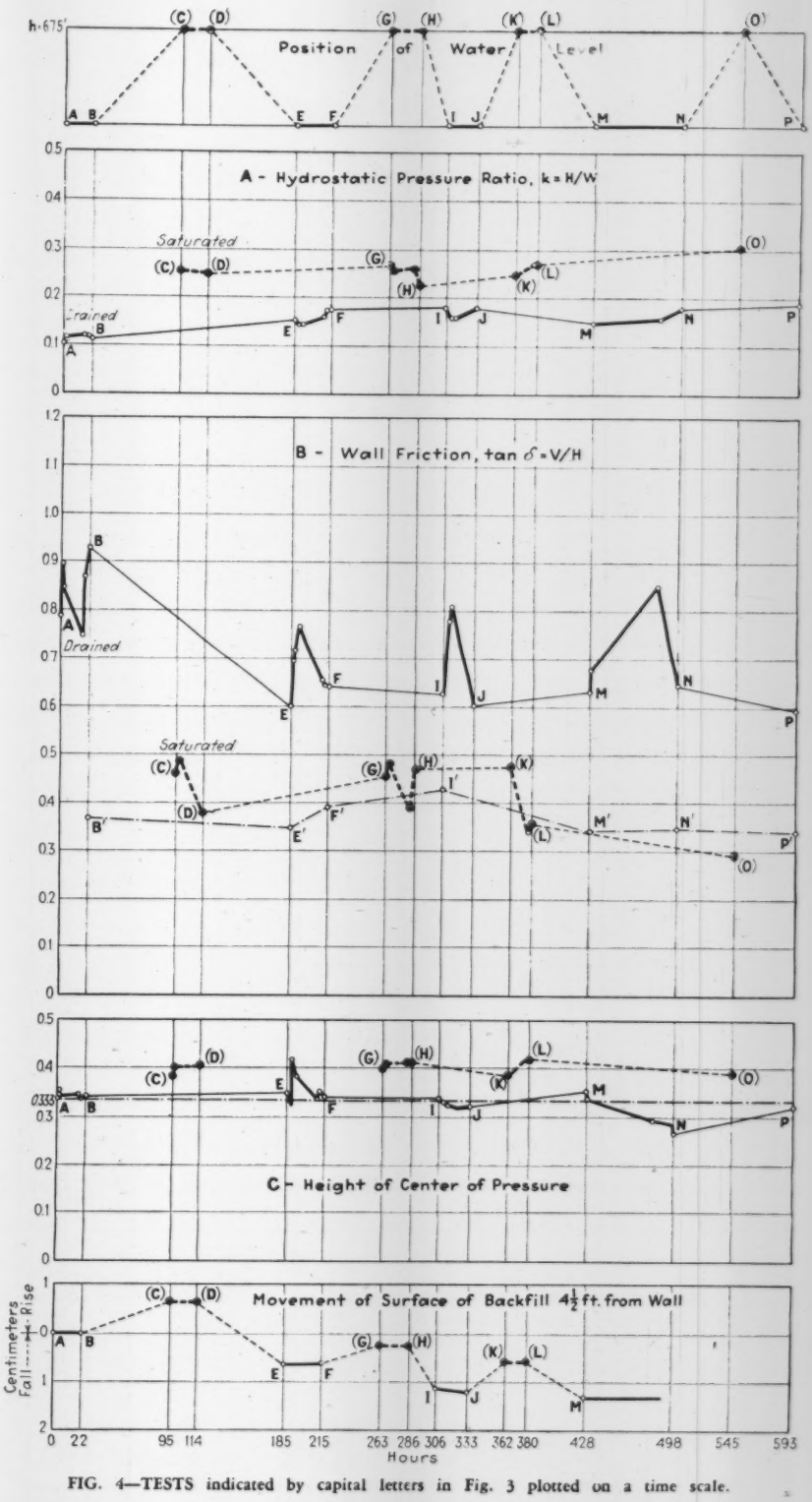

h=.675'
Position of Water Level
A - Hydrostatic Pressure Ratio, k = H/W
Saturated
Drained
B - Wall Friction, tan δ = V/H
C - Height of Center of Pressure
Movement of Surface of Backfill 4½ ft. from Wall
Centimeters Fall — + Rise
Hours
0 22 95 114 185 215 263 286 306 333 362 380 428 498 545 593

FIG. 4—TESTS indicated by capital letters in Fig. 3 plotted on a time scale.

Since the fissures closed at each submergence, the depth to which fissures opened was by no means the same for all the stages of the test. Nevertheless, for our computations we were obliged to assume that the open fissure extended in every case through the full depth of the suction zone. Under these circumstances the computation can be expected to indicate the magnitude of the quantities only approximately. It was made by

means of Eq. (1) to (3), assuming $m = 1$. The results are assembled in Table I. The abnormally high values of the capillary suction p for stages e and g (figures in parentheses) seem to indicate that in these stages almost no separation occurred between the fill and the wall.

From the remaining twelve figures we obtain an average value $p = 76$ lb. per sq.ft., with a scatter of ∓ 50 per cent. The total negative pressure W_s, which acted on the back of the wall during the drained state, therefore averaged $pbh = 7{,}300$ lb., against a value of $W_o = 20{,}700$ lb. for the positive water pressure which acted during the state of submergence. Hence $W_s = 0.35\ W_o$, against $W_s = W_o$ for a state of complete capillary saturation.

By introducing the values of p, Table I, into Eq. (3), one can compute tan δ for any of the drained states listed in the table, because all the other quantities contained in the equation are determined by direct measurement. The results of this computation will be found in Table II, which gives the average distance of the wall from its original position, the duration of the intermissions, the values of k and tan δ for the submerged states (letters in brackets), the corresponding values of the angle of internal friction ϕ computed by means of Coulomb's formula as described in Article I, and the corrected values of tan δ for the drained state computed by means of Eq. (3). The table shows that the correction eliminated the apparent contradiction between the values of tan δ for the submerged and the drained state.

To bring this fact clearly into evidence, the corrected values of tan δ were plotted in diagram B of Fig. 4, and the points B′, E′, ... thus obtained were connected by a dot-and-dash line. According to the figure, for the submerged state the values tan δ range for the part AP of the test between the limits 0.29 and 0.48, the uncorrected drained values between the limits 0.59 and 0.93, and the corrected values between the limits 0.32 and 0.44. In diagram B of Fig. 3 the range of scatter of the corrected values of tan δ for the last part of the test (AP in Fig. 4) is shown by the vertical distance between the points M′ and I′. The range is entirely within the scatter of the values tan δ for the submerged state. Hence the high values of tan δ for the drained state were merely due to an illusion, produced by ignoring at the outset the action of the capillary forces.

For the same reason the low position of the center of the lateral pressure for the drained state in diagrams C of Figs. 3 and 4 has nothing to do with the height of the center of the pressure exerted by the solid fraction of the backfill, because it is due to the combined effect of two different causes. There is no reason for assuming that the center of the pressure exerted by the solid fraction is lower for the drained state than for the submerged state. The latter consistently gave values close to 0.4 h. The same value was found for the height of the center of pressure for a tilting wall backfilled with dry sand, before first slip (Test 1 of Article I).

TABLE I—CAPILLARY SUCTION IN TILL BACKFILL

State		p, lb./ft.²	State		p, lb./ft.²
a	of Fig. 3	78	B	of Fig. 4	126
d		93	E		70
e		(185)	F		66
g		(160)	I		46
i		45	J		43
m		64	M		95
			N		98
			P		88

It should be noticed that state a, Table I, corresponds to the backfill prior to first saturation. At that state the water content of the fill ranged between 9 and 11 per cent, against 13.9 per cent for the saturated and 12.6 per cent for the drained state. Nevertheless, while the fill was being placed and compacted in a moist state, a negative hydrostatic pressure developed of the same intensity as the pressure which acted in the drained fill in the later stages of the test.

Effect of repeated drainage and saturation on angle of internal friction—The values of ϕ in Table II indicate that the outward movement of the wall had but little effect on the angle of effective internal friction, which is in striking contrast to what was observed during the tests with backfills consisting of sand. In a single case (at the end of movement j-k in Fig. 3) the value of the angle of internal friction temporarily went up to 39° 40′, but during the following 21 hours it returned to 29° 20′. We suspected that the apparently abnormal behavior of the till was due to the thorough readjustment of the grains and other structural changes produced by repeated drainage and saturation. Any such readjustment reduces the stresses in the fill and prevents them from assuming their maximum value.

In order to find out whether this explanation is correct, we repeated the test with the following modification: While filling the bin in 6-in. layers we kept the water in the standpipe about 1 ft. below the surface of the fill, but immediately after the bin was filled we raised the water level to the surface. After two days we drained the fill and left it in this state of partial capillary saturation until the end of the test. During the test the wall was moved away from the fill by tilting around a horizontal axis, in the same manner as in the preceding test, to an average distance of 10 mm. or 0.005 h from its original position. In diagrams A to C of Fig. 5 the results of the test are plotted in the same manner as in Fig. 3 of the preceding test—that is, the existence of the suction effect was disregarded. When the wall arrived at an average distance of 2.7 mm. from its original position, a crack appeared on the surface of the fill about 2.2 ft. back of the wall. Through the window one could see that this crack dipped away from the wall at an angle of about 70° with the horizontal and extended to a depth of about 0.8 ft.

To obtain the real values of pressure ratio and wall friction in Fig. 5, correction must be made for the capillary forces (negative hydrostatic pressure) which acted on the wall. If in the formula for capillary pressure, Eq. (1), we introduce the values $H_d = 9{,}890$ lb., $W_s = 42{,}900$ lb., $k = 0.304$, $s_s = 142.9$ lb. per cu.ft., $h = 6.55$ ft., and $m = 1$, which correspond to the conditions existing at the beginning of the test, prior to draining the fill, we obtain as the value of the negative hydrostatic pressure $p = 54$ lb. per sq.ft. But Eq. (4), applicable when wall and fill remain in contact, gives $p = 51$ lb. per

TABLE II—RESULTS OF RETAINING-WALL TESTS ON TILL BACKFILL

Outward Movement of Retaining Wall from Original Position, mm.	State of Backfill	Hydrostatic Pressure Coefficient, k	Coefficient of Wall Friction for Submerged State, tanδ	Angle of Internal Friction for Submerged State, ϕ	Coefficient of Wall Friction for Drained State (corrected) tanδ	Intermission, Hours
0	a, Fig. 3	0	0	0	0.07	
0	(b) Fig. 3	0.351	0.38	23° 0′	0	
0	(c) Fig. 3	0.381	0.30	22° 20′	0	……17
0	d, Fig. 3	0	0	0	0.26	
0.15	e, Fig. 3	0	0	0	0.30	
0.15	(f) Fig. 3	0.376	0.41	21° 20′	0	
0.15	g, Fig. 3	0	0	0	0.26	
0.45	i, Fig. 3	0	0	0	0.47	
0.45	(j) Fig. 3	0.258	0.55	30° 10′	0	
0.85	(k) Fig. 3	0.171	0.67	39° 40′	0	
0.85	(l) Fig. 3	0.273	0.49	29° 20′	0	……22
0.85	m, Fig. 3	0	0	0	0.37	
4.20	A, Fig. 4	0	0	0	0	
4.20	B, Fig. 4	0	0	0	0.37	……21
4.20	(C) Fig. 4	0.252	0.46	31° 40′	0	……20
4.20	(D) Fig. 4	0.247	0.38	33° 00′	0	
4.20	C, Fig. 4	0	0	0	0.35	……30
4.20	F, Fig. 4	0	0	0	0.39	
4.20	(G) Fig. 4	0.262	0.46	30° 50′	0	……24
4.20	(H) Fig. 4	0.222	0.47	34° 50′	0	
4.20	I, Fig. 4	0	0	0	0.43	……28
4.20	J, Fig. 4	0	0	0	0.44	
4.20	(K) Fig. 4	0.245	0.48	32° 20′	0	……17
4.20	(L) Fig. 4	0.271	0.35	31° 00′	0	
4.20	M, Fig. 4	0	0	0	0.32	
4.20	N, Fig. 4	0	0	0	0.35	……72
4.20	(O) Fig. 4	0.302	0.29	28° 50′	0	
4.20	P, Fig. 4	0	0	0	0.34	

NOTE: Parentheses indicate that the point corresponds to the submerged state of the fill. The intermission gives the time in hours during which the fill remained in drained or saturated state at stationary position of the wall.

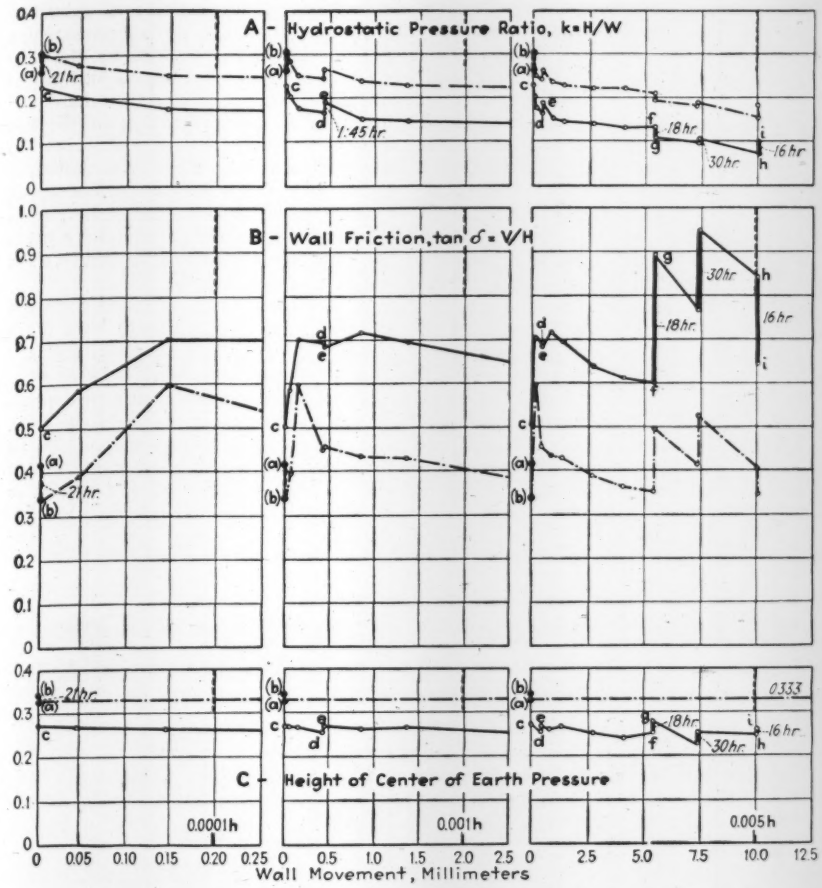


FIG. 5—RESULTS of test of till in drained state showed that a high value of internal friction was developed by wall movement.

sq.ft. For the computations the higher value was retained. It is somewhat smaller than the average value, which was obtained for p in the preceding test. Considering the scatter of the values assembled in Table I, the value $p = 54$ lb. merely represents an approximation, yet it may be used for a crude estimate of the values k and tan δ in the test Fig. 5.

Using Eq. (1) with the value $n = 1$, solving for k, and introducing the numerical values $W_s = 42{,}900$ lb., $h = 6.55$ ft., $s_s = 142.9$ lb. per cu.ft., and $p = 54$ lb. per sq.ft., as well as the measured values of horizontal pressure, results were obtained as plotted in A of Fig. 5 by a dot-and-dash line. Increasing the value of p from 54 to 76 lb. per sq.ft. (average value for the preceding test) would increase the computed values of k by only 0.026; hence an appreciable error in estimating the value of p has a rather small effect on the results of the computations.

Table III contains the corrected values of k, tan δ and ϕ for the test Fig. 5. This table demonstrates that the value ϕ consistently increases as the wall moves out, finally reaching a value as high as 43° 30′. Thus, when the fill is kept in drained state throughout the test, the outward movement of the wall mobilizes the internal frictional resistance of the fill in the same fashion as it does for a backfill consisting of dry sand. There is no doubt that the same result would be obtained if the fill were maintained throughout the test in a saturated state.

Subsidence due to repeated drainage and saturation—As often as the fill passes from the submerged into the drained state, the vertical pressure transmitted through the grains of the backfill increases considerably, due to the elimination of hydrostatic uplift and due to the surcharge produced by the surface tension of the water. This increase of the pressures should cause a compression of the fill and subsidence of its surface. On the other hand, a submergence of the drained fill should cause a rise due to elastic expansion of the fill. These conclusions are confirmed by the diagram at the bottom of Fig. 4c, which shows the upward and downward movement of the surface while the wall was stationary at an average distance out of 4.2 mm. According to the diagram, the rise of the surface due to submergence is always a trifle smaller than the preceding drop due to drainage, which indicates that each drainage produced a slight permanent increase of the density of the fill.

At the outset of the test, when the wall was still in its original position, a submergence-drainage cycle caused a practically irreversible lowering of the surface of the fill through a distance of 0.5 in. The total permanent subsidence prior to state A, Fig. 4, was about 0.9 in. However, at each repetition of the process of drainage, the difference between rise and fall became smaller, so that probably after many repetitions (with wall held stationary) the rise would become equal to the preceding drop.

Comparing the subsidence of the till in the first cycle with that of the sand (Article II), we notice the following facts:

	Rise and Fall of Surface	
	Till (Test Fig. 3)	Sand (Art. II)
First saturation ..	—0.5 in.	+0.003 in.
First drainage	—0.25 in.	—0.02 in.

Thus, though the initial volume of voids of the till was 28 per cent against 40 per cent for the sand, yet the till was much more affected in its structure by the circulation of the water through its voids than the sand. Further testimony for the deficient stability of the structure of the till can be found in Fig. 4, which shows the variations of pressure ratio, wall friction and height of center of pressure while the wall was held at an average distance of 4.2 mm. from its original position. Each one of the consecutive stages AB, CD, etc., represents an intermission during which the external conditions of the fill remained unchanged; nevertheless none of the quantities plotted remained constant during the intermissions, which indicates the occurrence of minor grain movements within the fill.

Due to the fact that the fill had been subjected to repeated cycles of drainage and saturation prior to state A, Fig. 4, these changes merely represent more or less erratic deviations from average values, without any tendency toward a

TABLE III—EFFECT OF WALL MOVEMENT ON MODIFIED-TILL BACKFILL

Outward Movement of Retaining Wall from Original Position, mm.	State of Backfill	Hydrostatic Pressure Coefficient, k	Coefficient of Wall Friction, tanδ	Angle of Internal Friction, ϕ	Intermission, Hours
0	c	0.304	0.33	28° 0′	
0.42	d	0.247	0.44	32° 30′	} 2
0.44	e	0.267	0.46	30° 10′	
5.39	f	0.209	0.36	37° 20′	 19
5.40	g	0.194	0.50	38° 0′	
10.20	h	0.157	0.41	43° 30′	 16
10.20	i	0.182	0.35	40° 40′	

permanent deviation. Yet these average values are very much smaller than the possible maximum values. In Table II the average values are tan δ=0.39, δ=21° 20′, and ϕ=31° 50′, while Table III gives the maximum values as tan δ=0.50, δ=27° 0′, and ϕ=43° 30′. Since friction tests merely inform us about the maximum values of δ and ϕ, it is obvious that with fine-grained backfills ample allowance should be made for the effect of subsequent structural change on the coefficient of friction.

The first drainage of the backfill in the test Fig. 3 caused a subsidence of 0.25 in. against 0.14 in. for the test Fig. 5. Hence wet placing of the fill (test Fig. 5) was found to produce a more stable arrangement of the grains than dry placing as used in the test of Fig. 3.

The following conclusions were reached as a result of these studies:

1. When the backfill was drained, a negative hydrostatic pressure developed in the water retained by the fill. This pressure pulled the wall toward the fill with a force ranging between 50 and 120 lb. per sq.ft. A similar negative pressure (suction) developed while placing and compacting the fill in 6-in. layers.

2. When draining the backfill, an open fissure is likely to be formed between the upper part of the fill and the back of the wall. In this case the lateral pressure of the upper part of the fill is taken up by the surface tension of the water, and the wall is subject to pressure from the lower part of the fill only.

3. For both the submerged and the drained state, the lateral pressure exerted by the water and by the solid fraction of the fill must be considered independently.

4. Repeated drainage and submergence partly release the stresses which are set up in the backfill by the outward movement of the wall and consequently increase the lateral pressure exerted by the solid fraction of the fill. For the final state of equilibrium to which this process leads at a stationary position of the wall, the coefficients of wall friction and of internal friction were found to be approximately equal to two-thirds the maximum values which they can assume.

5. A till that is in saturated condition at time of placing seems to have more stable structure than till that is placed and compacted in a moist state.

Previous articles in this series appeared in our issues of Feb. 1 (p. 136), Feb. 22 (p. 259), March 8 (p. 316) and March 29 (p. 403). A later article will deal with application of the results to actual design.

New Velocity-Measuring Device Developed in Waterways Laboratory

A VERY sensitive instrument for measuring small velocities in flowing water has been developed in the U. S. Waterways Experiment Station, Vicksburg, Miss., by Carl E. Bentzel, research assistant at the station, in connection with the studies of the transportation of bed-load material being carried on at the station under the direction of Lieut. H. D. Vogel. The following description of the Bentzel velocity tube is taken from Paper No. 17 of the U. S. Waterways Experiment Station which is to be published in June.

The new instrument is designed to measure velocities ranging from 0.10 to 5 ft. per sec., and has been found so sensitive and ideally suited for velocity measurements at the station as entirely to supersede the use of Pitot tube and other velocity meters.

The arrangement of the velocity tube is shown in Fig. 2, and one of the tubes in use is shown in Fig. 3. The water flowing into the upstream leg of the tube causes a velocity head to be created. The velocity head on the downstream leg, on the other hand, is negative. This difference in head causes the circulation through the tube of a small quantity of water, the amount depending upon the velocity of the water flowing in the flume or pipe into which the tube has been lowered. Inside the right-hand leg of the tube, which has an even taper inside, is a small float made of a piece of capillary glass tubing, closed at both ends. This float has been built to have very slight buoyancy.

When there is no flow through the tube, i.e., when the velocity being measured is zero, the float rises until it rests against a wire stop in the upper end of the tapered tube. When water is flowing through the tube, however, the impact of the flowing water causes the float to be pushed down the tapered tube. At

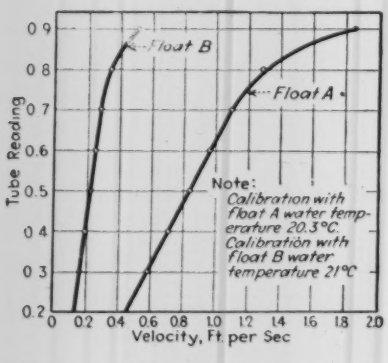


FIG. 1—RATING CURVE for floats used in the velocity-measuring tube.

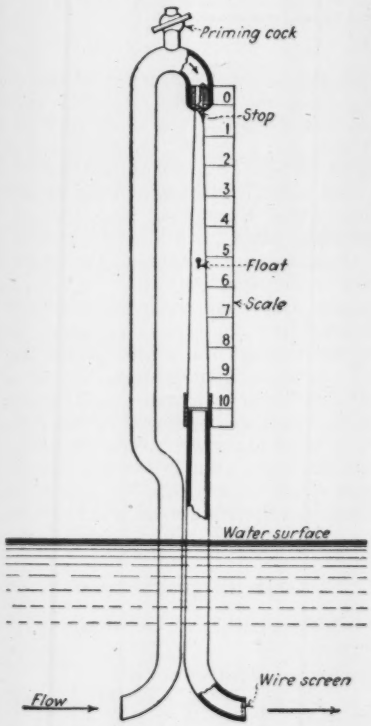


FIG. 2—DETAILS of the arrangement of the velocity-measuring tube developed at the U. S. Waterways Experiment Station.

FIG. 3—BENTZEL VELOCITY TUBE being used in a model under study at the Vicksburg laboratory.

some point within the length of the tapered section, the impact force of the water, reduced by the enlarged section, exactly balances the buoyancy force of the float, causing it to come to rest.

It has been found that this instrument can be calibrated very closely by towing through still water, and that for every velocity of flow, within the range of the instrument, there is a corresponding position of the float within the tapered section. Thus rating curves may be constructed from which velocities may be read directly.

By the use of floats of various specific gravities and combined with tapered tubes of varying inside dimensions, almost any velocity can be measured with one of these tubes. Necessarily, the effect of temperature on the viscosity of the water must be considered. Fig. 1 shows the calibrations for two of the floats used in the bed-load tests at the Vicksburg Laboratory.

Mr. Bentzel, the inventor of the instrument, is a graduate of Chalmers Institute of Teknologi, Gothenburg, Sweden, and has been employed as a research assistant at the Waterways Experiment Station for the past two years. He has applied for a patent on the velocity tube with the reservation by the U. S. government of the right to manufacture the instrument for its own use, since it was perfected at the Waterways Experiment Station.

Remedy of Frost Heaves on Highways

Survey of practice and of results in prevention of frost heaves on various road surfaces with various characters of subsoil profiles

A SURVEY OF CAUSES AND REMEDIES for frost heaves in road surfaces has recently been summarized in *Public Roads* by Henry Aaron, assistant highway engineer, division of tests, Bureau of Public Roads. Being strictly a record of soil profiles and of the actual corrective measures applied, the survey has high practical interest. Its essential parts are summarized here. —EDITOR.

THE HEAVING produced in various types of soils that form the foundation for road surfaces has been studied extensively, both in the laboratory and in the field. Many kinds of remedial treatments have been undertaken by different engineers. To correlate the various methods, the Bureau of Public Roads with the cooperation of the highway departments of Minnesota, Wisconsin and Michigan conducted a survey from 1928 to 1933 to determine for specific cases: (1) the method of treatment used; (2) its performance under service conditions; and (3) the physical properties of the subgrade soils in the location and their arrangement in the soil profile.

No attempt is made in this report to explain the physics or mechanics of frost heaving. It is intended, primarily, as an exposition of methods used to prevent harmful frost action and the results of these methods. To interpret properly the results of the various methods, it is important to know the characteristics of the different subgrade soils that make up the soil profile in the location where they were used.

Soil profiles

Section A, Fig. 1, illustrates pockets of silt and silty clay varying in shape and depth within a deposit of porous sandy soils. The heaving in the silt pocket is excessive, while that in the surrounding sandy soils is negligible. Considerable heave occurs also in the pocket of silty clay. However, it is covered by an appreciable layer of sand (about 18 in.), which reduces the effect on the surface.

Section B shows a typical soil profile in the loessial area of Minnesota and Wisconsin. The frost heave is confined to that portion of the road resting on the unweathered or slightly weathered structureless silt. The weathered upper layers of the profile are granular in structure and apparently do not suffer detrimental frost heave. The structureless loessial soil is a silt or silt loam containing a high percentage of very fine sand, has a high water-holding capacity, is unstable when wet and possesses capillary properties in a high degree. Resting on a relatively impervious clay that retards percolation and forms a water table at its base, a condition is created favorable to strong functioning of the high capillary properties of the structureless silt layer. The clay may be at a considerable depth below the road surface and is not necessarily in as close proximity to the surface as is indicated by Section B.

An example of detrimental frost heave in a sandy soil is illustrated in Section C. The sand, which varies from fine to coarse in texture, is maintained in a saturated condition by a water table close to the surface. This is due largely to the topographic features of the adjoining terrain. The road cuts through small knobs or ridges, interspersed with bogs located both at higher and at lower elevations than the roadway.

A soil profile in extremely variable glacial materials is illustrated in Section D. The relation between frost heave and the various layers of the soil profile is similar to that described for B. In this case the heaving occurs in an unweathered glacial till containing lime, sand pockets and stratified sand and silt. Water is transmitted to the roadway by the capillarity of the glacial till and also by the stratified sand and silt. Considerable amounts of water collect in the sand pockets. Such variations within a soil layer are especially productive of differential frost heaving.

Section E is an illustration of a soil profile in which heaving occurs as a result of depressions in the surface of an impervious clay. The dense clay restricts percolation of water and maintains the overlying sand in a wet condition, while the sags in the surface of the clay layer act as reservoirs for the collection of water in excessive amounts.

Sections F and G are examples of soil profiles in which heaving results. The profiles consist of a surface layer of more or less unstable soil underlain by silts and very fine sands which occur in pockets and layers of varying thicknesses adjacent to water-carrying soil layers.

Sections H, I and J illustrate profiles that produce the greatest and most dangerous heaving. Stratified silt and very fine sand invariably heave in such amounts as to rupture any type of superimposed road surface. Very dangerous differential heaving is produced when soil layers subject to detrimental heave in varying amounts are arranged in pockets as in L and J.

A different type of heaving condition is shown in Section K. In this profile it includes very thin layers of silt which assist in keeping the clay very wet and soft.

The majority of the clay-loam soils similar to those shown in Section L are subject to detrimental frost heave only when wet and poorly drained, but considerable heaving has been observed in fine sandy clay loam containing appreciable amounts of disseminated lime.

Where the road surface rests on lime-free clay loam, no serious damage has resulted.

The detrimental results of lime accumulations in a soil layer have been observed also in soil profiles where a limey clay loam was found. Two adjacent clay-loam soil layers in the same road cut were found to possess similar physical properties, but the one containing lime heaved excessively, while the layer free of lime did not heave enough to damage the road surface.

The soil profiles illustrated in Sections M and N produce heaving similar to that of the stratified soils previously described and need no further discussion.

Differential heaving is certain where there is the extreme variation in soils

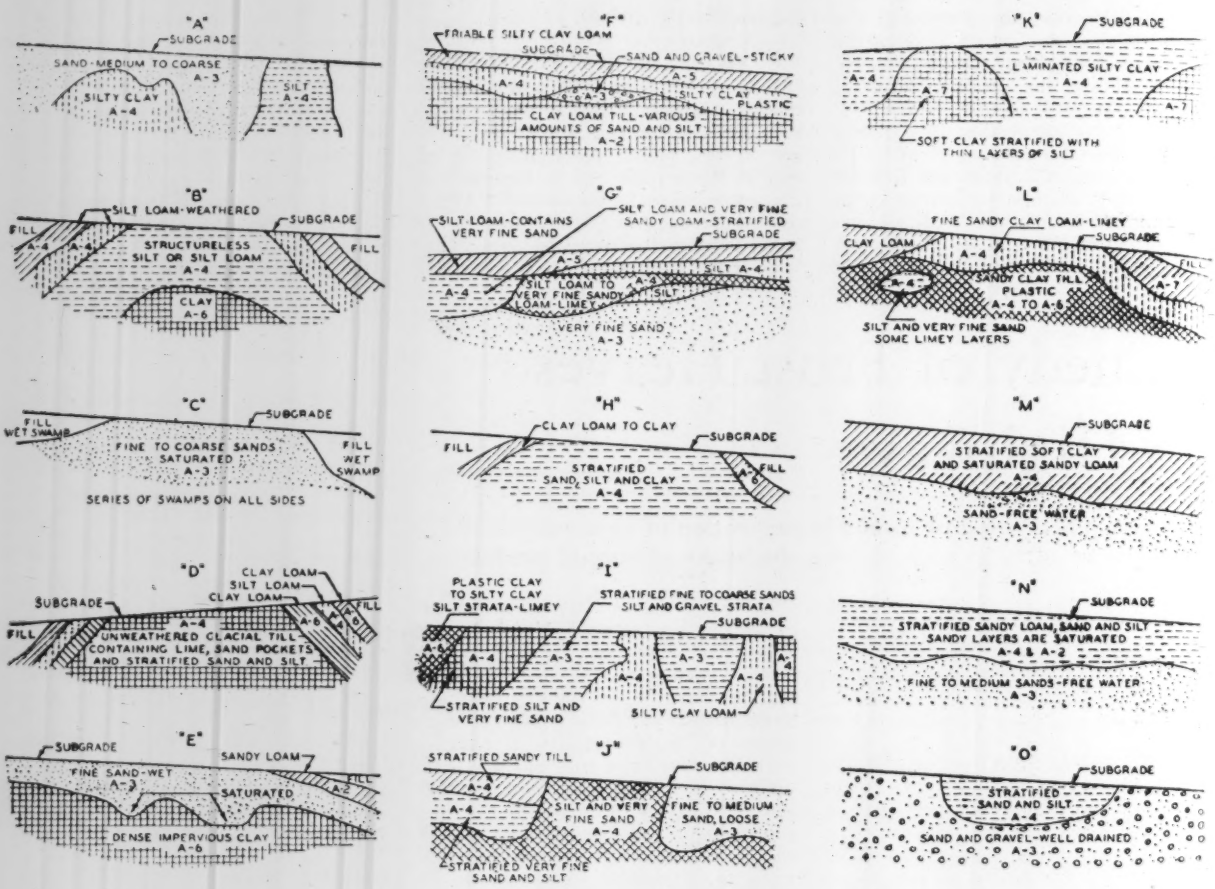


FIG. 1—SOIL PROFILES in which frost heaves detrimental to road surfaces have been observed.

as shown in Section O. The pocket of soil subject to frost heaving is composed of strata of water-bearing sand and strata of silt which absorb water readily and resist all attempts at drainage. The sand and gravel surrounding this pocket is well drained and does not permit the accumulation of water, and no heaving occurs in the sand and gravel.

Preventive measures

Various methods used by different engineers in efforts to prevent frost heaving are shown in Fig. 2. Considering only the locations inspected in this survey, A shows a method that has been used since 1921; B, C and D since 1928; E, F, G, H, I and J since 1929; and K to P, inclusive, since about 1920.

The methods of excavating heaving soil and substituting a non-heaving material are illustrated in A to I inclusive; J shows excavation of the heaving soil, treating the bottom of the excavation with bituminous material to cut off capillary action and replacing the excavated material. Drainage channels to intercept and direct water away from the roadbed are shown in K to P inclusive.

Generally no more than two years is required for the surface condition to reflect the effectiveness of the preventive method used. This is especially true of methods that do not serve the requirements of the location. In such cases heaving has been observed during the first winter after installation. Therefore, the results of methods that have been in service since 1929 are considered fully as significant as those that have been in service since 1920.

The performance of these preventive measures employed in connection with different types of road surfaces and the associated local soil condition is described as follows:

The narrow center trench types A, B and C serve to eliminate only that heaving which might occur directly over the area of the excavation. The soil on the sides heaves sufficiently to crack concrete and bituminous pavements. The damage to gravel surfaces caused by the frost uplift is ironed out by maintenance after the frost leaves the ground, so that there is no visible aftermath of the winter condition. These methods reduce the amount of break-up during the spring thaws and allow passage of vehicles. Compared with the results where no precautions are taken, the benefits of this method are considerable.

Treatment for the full width of the traveled area, as illustrated by type D, has proved advantageous. There was no noticeable difference between the riding qualities of the surface laid over this type of treatment and adjacent sections where heaving had always been absent. The entire roadway remained smooth and firm during the spring thaw.

The V-type trench (type E) is very popular because it can be excavated with a blade grader, which materially reduces the cost of excavation. In most cases surfaces laid over subgrades so treated carry traffic in a satisfactory manner. However, the heaving at the sides of the trench 12 ft. wide in soil profiles D and J and the general heaving in the trench 26 ft. wide in soil profile I has, in some cases, developed to the point where driving is safe only at very low speeds. The surfacing in the heaved area is badly broken.

Measures F, G, H and I have proved satisfactory and reliable methods of preventing detrimental frost heave of concrete pavements. Except under the extreme climatic conditions of the northern parts of Michigan and Wisconsin, heaving is negligible in treatments where the excavation and backfill are not less than 2 ft. Noticeable but uniform heave has been measured in treatments 1 ft. deep.

The method of excavating the heaving material for the entire width of the roadway, treating the base of the excavation

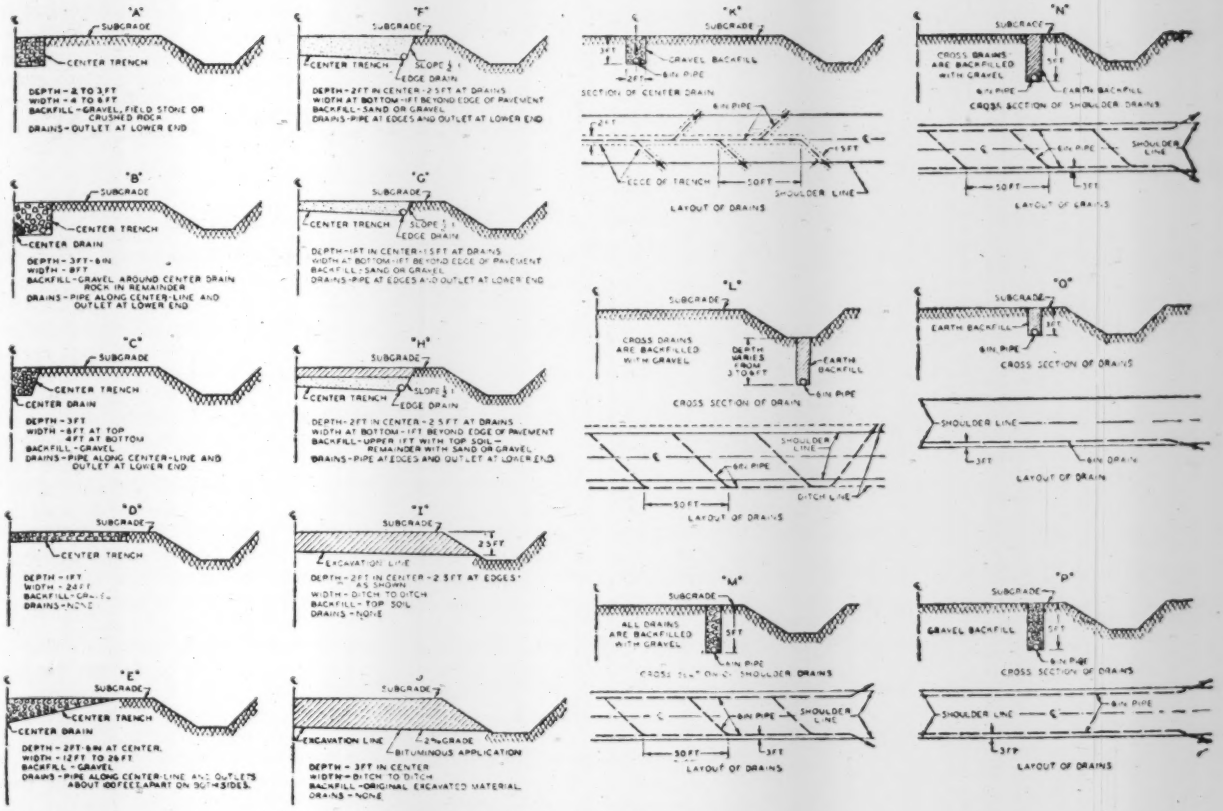


FIG. 2—SUBGRADE DESIGNS undertaken to reduce frost heaves in types of soil profiles shown by Fig. 1.

with a bituminous material and replacing the excavated material (type J) has not been quite as successful as the full-width treatments described above. Gravel surfaces on bituminous-treated soils have become uneven and rough and cut up during the spring thaw. The heaving of a concrete pavement is noticeable but fairly uniform. Cracking is more pronounced than in the locations where a selected backfill material was substituted for the excavated frost heaving soil.

The various kinds of treatment should not end abruptly but should be tapered or feathered out. According to observations by Andrew Seifert, concrete pavements laid on gravel backfill tapered from 10 to 25 ft. cracked considerably and became noticeably rough; 50-ft. tapers have been more satisfactory.

The drainage types L to P, inclusive, have been of practically no benefit in preventing heaving of soils. In most cases the drains have been placed at arbitrary depths of 3 to 6 ft. without regard to the type of soil or the arrangement of the soil layers. Type M, when installed in water-bearing sandy soils (soil profiles C and E), in such a manner as to conform to the soil profile, served to intercept and carry away enough of the water to prevent detrimental heave. This same type, when used in the stratified silts, fine sands and clays (soil profile I), was of no value whatever.

However convenient it would be to be able to establish definite and invariable spacing and depth requirements for drains, the hopelessness of thus standardizing drainage design becomes increasingly apparent as research in soils, especially for engineering purposes, progresses.

Failure to obtain the desired stability is definitely attributable in many instances to the attempt to apply arbitrary standards of spacing and depth in the placing of the drains without regard to prevailing conditions of profile or soil characteristics.

There is only one practical procedure, and that is to place drains at such distances apart and at such depths as the interception and removal of offending moisture require. And this, it can readily be seen, becomes a local problem, the solution of which depends upon such factors as the source of the water to be removed, the character of the soils comprising the different layers and the arrangement of the layers in the soil profile.

To complicate the problem further, some soils very readily give up their contained water and thus may be easily stabilized by drainage; other soils exercise a high affinity for moisture and are not apt to be stabilized by drainage. The only profiles found in this survey in which drainage can be reasonably certain to prevent frost heave are shown in Fig. 1, C and E.

Fig. 3A shows how drains should be placed in order to prevent frost heave in soil profiles of the type illustrated in Fig. 1E. Frost heave occurring in drainable soils due to a permanent high water table (Fig. 1C) may be eliminated by the system of drains shown in Fig. 3B. It should be borne in mind that these methods are applicable only where the soil is more or less porous and does not possess capillarity in an appreciable amount.

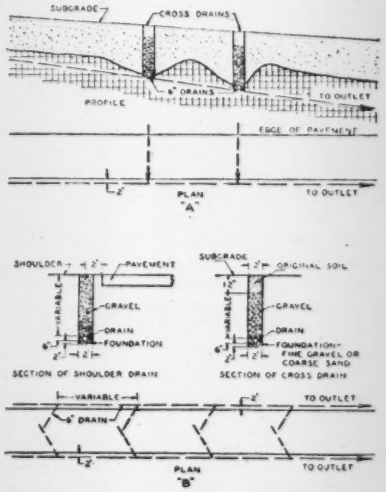


FIG. 3—DRAIN DESIGN to prevent frost heaves in drainable soils.

Ten Business Problems of Contracting

X—Management of Construction Labor

In modern mechanized operations management of labor consists in creating a better balance between the forces assigned to the various operations

By J. L. Harrison
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THE MANAGEMENT of construction is largely a matter of the use of labor. Accordingly, success in this field depends in large part on the success with which employment and output can be harmonized. Not so many years ago this harmony was secured by driving the men. In those days the most successful overseer was the man who could drive his men the hardest without driving them off the job. Little equipment was used. Complex processes were all but unknown. The idea of balanced operations lay in the future. In short, production was measurably a matter of brute strength. Little was known of organization as a means of securing efficiency, and less was practiced. "Pound 'em over the backs, Jerry, pound 'em over the backs" was the general rule. It was hard on the men, but it did get results. Perhaps that was all the justification it required. In any event that is the only justification it had.

Production by organization

Today the management of construction is quite different. Equipment has replaced a large fraction of the labor formerly used. Processes are more complex. In most lines of work more interdependent processes are used. Many series of interdependent processes are long and rather intricate. In a good many of these series technical or other considerations prevent more than a sharply limited amount of storage between processes. These and other considerations make production more and more a matter of organization—less and less a matter of driving the men.

The reference just made to construction as a series of successive interdependent operations deserves some further comment, for this aspect of construction does not appear to be too well understood. As an illustration of a series of successive interdependent operations, the series used on a power-shovel highway-grading job will suffice. Ordinarily this series would be about as follows:

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Operation 2—The hauling unit hauls it to the fill and dumps it there.

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Operation 4—The rollers compact these layers.

Now in any series of this sort—and most construction work involves one or more of them, often far more complex than the one just given—it is apparent that if the best possible output and the lowest cost are to be secured, there must be enough hauling units to move all of the dirt that the shovel can dig. There is no storage between the shovel and the hauling units. The shovel cannot dig today for use tomorrow or dig at a steady rate at any time or for any period unless there are hauling units to move the diggings. These successive operations are, then, absolutely interdependent. The shovel can do nothing without the hauling units, and these can do nothing without the shovel.

When the hauling units reach the dump the same interdependent relationship is encountered. The trucks will not be permitted to dump more material than the spreading unit can spread correctly, and the spreading unit will not be permitted to spread material faster than the rollers can compact it. No storage of consequence can exist between any two of these processes. Therefore, they are, as it were, rigidly interlocked. For one to function all must function. If one falls behind, all fall behind. If one stops, all stops. In short, this is a typical series of successive interdependent or interlocking processes, and as such it is illustrative of the situation that exists in most modern construction work.

But wherever production depends on a series of successive operations such as this, it is obvious that in so far as the series is rigidly interlocking, full production can be secured only when all processes can support the full output capacity of the principal unit—in this case, the shovel—and that the lowest unit cost can be reached only when the output capacity of all processes is in exact harmony with the full output capacity of this unit and each is properly manned.

The general principal thus briefly outlined, lays emphasis on two facts. The first of these is that high rates of production and their corollary low-unit production costs, depend on a thorough appreciation of this basic view of construction and on the accurate interpretation of it in the design and in the management of construction organizations.

The second of these two facts is that where construction is obtained through a series of interlocking processes not much of anything can be more futile than attempts to speed up the work by speeding up the men. They have relatively little to do with the rate of production. All they can possibly do that is of any value is to perform the work that there is to do. If the shovel is working badly, "pounding" the men on the dump will not speed it up. If there are not enough trucks, "pounding" these men will not supply them. If the job is short a roller, only the job management can correct this deficiency. In short, when production is low the cause usually lies either in the design or in the operation of the organization. The responsibility for it lies with the management. What, then, can be more futile than "driving" men on a modern construction job? To do so produces no valuable results and is pretty certain to ruin morale. The superintendent who attempts it should be eliminated as plainly lacking an appreciation of the responsibilities of the position that he holds and wholly out of step with the times.

As a matter of fact, labor shows little tendency to shirk. Even in good times the willingness of men to do a full day's work, to do it faithfully and to do it well has been a conspicuous fact on practically all of the rather considerable number of jobs that the writer has been privileged to study. Within their reasonable capacity, workmen stand ready to do the work that it falls to their lot to do. And just here lies the next important point in successful construction management. In his design of the construction organization it is the superintendent's responsibility to see that the right amount of work can reach and does reach each man on the job. The man himself cannot go out and find this work. He is assigned to a specific task. That task he must and will do. But it is no responsibility of his if the task is lighter than it ought to be. Nor can he correct this condition. Obviously, the primary approach to correction can only lie with the job management and can be effectuated by the job management only by creating a better balance between the forces assigned to the various operations which comprise the production series.

This presents another aspect of job management. It has already been remarked that when production results from a series of successive interlocking processes, all should, in theory, have equal output capacity. Any defect in the design of a construction organization caused by a failure to recognize this fact will have either of two results. If some process has too little output capacity, production for the series as a whole is correspondingly reduced. If some of the processes are overdeveloped, unit cost is increased.

Proper balance required

The output capacity of any process in an interlocking series usually involves both equipment and personnel, though scattered throughout the construction field there still are a good many processes that involve only hand labor and hand tools. In general, then, if the output capacity of the various processes in an interlocking series is to be held in balance at any output level, it is necessary to consider each from the standpoint of equipment capacity and labor requirements. If this is done and, as a result of such consideration, the correct amount of labor as well as the correct amount of equipment is assigned to each process, the proper work load can be made to reach each laborer.

The point to be emphasized here is that there should be nothing haphazard about the design of a construction organization. Each part of it has a load to carry which is as definitely determinable in advance as the load each member of a bridge truss will carry. This having been determined, the means of carrying it—that is, the machines and the men it will be necessary to provide—may be worked out. It is only in this way that the construction manager can provide a proper load for each workman and make certain that all labor is in a position to have and so to do a full day's work. Once this has been done, there will be little trouble in getting a full day's work out of each man.

When this matter is approached in this way, it is apparent that the first and probably the most essential step toward keeping the amount of labor used in harmony with the amount of work done, has been taken when the organization is so designed that a full day's work should reach each laborer every day. But this, of itself, does not insure proper production or minimum labor costs. After the organization has been designed, it must be built; and after it is built, it must be set in motion and kept running at the proper speed.

The thought that an organization must be built—that is, developed—alludes to a fact which may be expressed in the military axiom that a thousand men do not make a regiment. As a rule a good many of the men employed on a construction job are raw recruits. These, at least, must be trained to function as units in an organization. Many of them must be shown how to perform the tasks assigned to them. Some will be found unwilling or unfit and must be replaced. This process may continue for some time. However, it is a necessary process, for high production requires a well-integrated organization, and a well-integrated organization implies both that each individual is familiar with his work and that he is disciplined to thorough attention to it.

In practice it does not seem to be as difficult to secure disciplined attention to an assigned task as it is to train men to handle a task correctly. The most expert superintendent in the writer's acquaintance gives personal instructions to each man hired, not only as to what he must do but even as to the details involved in doing it. These details involve even such matters as how a hand tool must be held and the motions to be used in working with it. To many this may seem like needless attention to detail. The fact remains that painstaking attention to such details as this not only results in relieving workmen of a surprising amount of unproductive effort but in the long run measurably reduces the force required on processes on which most of the work is performed by hand labor.

It will, of course, be recognized by all contractors that there is a definite limit to the amount of effort a man can make day after day. If more or less of this effort is dissipated on needless and useless motions, the amount available for useful work is correspondingly reduced. The objective in training men to handle their tasks correctly is to reduce the waste of energy involved in the ordinary use of tools, and thus to make available a larger amount for useful work. A great deal more attention to this phase of building efficient construction organizations is warranted.

Capacity speed essential

Having built a good organization and set it in motion, the next problem is to bring it to proper speed and keep it running at this speed. Here the equipment, rather than the labor, normally dominates the situation. Take a concrete paving job as an illustration. The organization is designed to produce 120 ft. of pavement an hour. Production involves a complex series of rigidly interdependent operations. The organization is in excellent balance throughout and well disciplined. But the mixer is not very dependable, and the crane gives trouble every now and then. The objective is 120 ft. of pavement every hour, ten hours every day. To reach this objective requires one thing, perfect equipment. Obviously the objective is unattainable on any other basis. The value of dependability in equipment can be illustrated in no better way. Still this is not the important point here. Rather, the point is that having designed and built a good organization and set it in motion, labor costs will still not be kept at a minimum unless proper attention is given to the factors that are essential in keeping it in motion at full speed.

The most essential of these factors is dependable equipment. Indeed of the half-dozen most essential factors, at least the first five are dependable equipment! Without it time is lost—a little here and a little there—but in total often a good deal. Production suffers correspondingly, for production is not a thing that can be speeded up at will. The organization is built to maintain a given rate. It cannot function at a higher rate. Lower rates and lost time are a dead loss.

Foresight important

Managerial foresight probably stands next in importance in its general effect on production. Few jobs run, as it were, on the dead level. Conditions on the job change from day to day. To meet these changing conditions, the organization must be subject to frequent readjustment. To keep production up to standard, the capacity of each process must be kept up to standard. But the load that falls on a given process—as an illustration, the load that falls on the subgrade crew on a paving job—will change a good deal from day to day. These changing conditions must be foreseen, and provision made to meet them before they are encountered.

This is, of course, only a single illustration of the thousand and one things that come up during the life of a construction job, which, if foreseen, will give no trouble because adequate provision for meeting them can be made, but which, if not foreseen, are certain to reduce production. Foresight, coupled with the ability to act when action is required, is essential to success.

While there are a good many other factors that might be mentioned as having an influence on the maintenance of a high uniform rate of production, only one other—managerial efficiency—will be mentioned. By this term the writer refers to the ability of the construction manager to secure the thing—in this case, production—that he knows he ought to have. It matters not whether this is termed force of character, personality or whatnot. To keep a job running at proper speed, the manager must know what is a proper speed and then see that that speed is maintained. Often it is a great deal easier for subordinates to make excuses than it is to make yardage. The easy-going superintendent accepts the excuses, the man who has force of character demands and gets the yardage.

All of the matters mentioned above are important as a means of keeping the amount of labor employed in harmony with the production secured. If they are properly handled, the amount of labor used will harmonize with the production secured. But their virtue is by no means limited to their effect on the amount of labor used. On many types of construction the costs generated

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The thought that an organization must be built—that is, developed—alludes to a fact which may be expressed in the military axiom that a thousand men do not make a regiment. As a rule a good many of the men employed on a construction job are raw recruits. These, at least, must be trained to function as units in an organization. Many of them must be shown how to perform the tasks assigned to them. Some will be found unwilling or unfit and must be replaced. This process may continue for some time. However, it is a necessary process, for high production requires a well-integrated organization, and a well-integrated organization implies both that each individual is familiar with his work and that he is disciplined to thorough attention to it.

In practice it does not seem to be as difficult to secure disciplined attention to an assigned task as it is to train men to handle a task correctly. The most expert superintendent in the writer's acquaintance gives personal instructions to each man hired, not only as to what he must do but even as to the details involved in doing it. These details involve even such matters as how a hand tool must be held and the motions to be used in working with it. To many this may seem like needless attention to detail. The fact remains that painstaking attention to such details as this not only results in relieving workmen of a surprising amount of unproductive effort but in the long run measurably reduces the force required on processes on which most of the work is performed by hand labor.

It will, of course, be recognized by all contractors that there is a definite limit to the amount of effort a man can make day after day. If more or less of this effort is dissipated on needless and useless motions, the amount available for useful work is correspondingly reduced. The objective in training men to handle their tasks correctly is to reduce the waste of energy involved in the ordinary use of tools, and thus to make available a larger amount for useful work. A great deal more attention to this phase of building efficient construction organizations is warranted.

Capacity speed essential

Having built a good organization and set it in motion, the next problem is to bring it to proper speed and keep it running at this speed. Here the equipment, rather than the labor, normally dominates the situation. Take a concrete paving job as an illustration. The organization is designed to produce 120 ft. of pavement an hour. Production involves a complex series of rigidly interdependent operations. The organization is in excellent balance throughout and well disciplined. But the mixer is not very dependable, and the crane gives trouble every now and then. The objective is 120 ft. of pavement every hour, ten hours every day. To reach this objective requires one thing, perfect equipment. Obviously the objective is unattainable on any other basis. The value of dependability in equipment can be illustrated in no better way. Still this is not the important point here. Rather, the point is that having designed and built a good organization and set it in motion, labor costs will still not be kept at a minimum unless proper attention is given to the factors that are essential in keeping it in motion at full speed.

The most essential of these factors is dependable equipment. Indeed of the half-dozen most essential factors, at least the first five are dependable equipment! Without it time is lost—a little here and a little there—but in total often a good deal. Production suffers correspondingly, for production is not a thing that can be speeded up at will. The organization is built to maintain a given rate. It cannot function at a higher rate. Lower rates and lost time are a dead loss.

Foresight important

Managerial foresight probably stands next in importance in its general effect on production. Few jobs run, as it were, on the dead level. Conditions on the job change from day to day. To meet these changing conditions, the organization must be subject to frequent readjustment. To keep production up to standard, the capacity of each process must be kept up to standard. But the load that falls on a given process—as an illustration, the load that falls on the subgrade crew on a paving job—will change a good deal from day to day. These changing conditions must be foreseen, and provision made to meet them before they are encountered.

This is, of course, only a single illustration of the thousand and one things that come up during the life of a construction job, which, if foreseen, will give no trouble because adequate provision for meeting them can be made, but which, if not foreseen, are certain to reduce production. Foresight, coupled with the ability to act when action is required, is essential to success.

While there are a good many other factors that might be mentioned as having an influence on the maintenance of a high uniform rate of production, only one other—managerial efficiency—will be mentioned. By this term the writer refers to the ability of the construction manager to secure the thing—in this case, production—that he knows he ought to have. It matters not whether this is termed force of character, personality or whatnot. To keep a job running at proper speed, the manager must know what is a proper speed and then see that that speed is maintained. Often it is a great deal easier for subordinates to make excuses than it is to make yardage. The easy-going superintendent accepts the excuses, the man who has force of character demands and gets the yardage.

All of the matters mentioned above are important as a means of keeping the amount of labor employed in harmony with the production secured. If they are properly handled, the amount of labor used will harmonize with the production secured. But their virtue is by no means limited to their effect on the amount of labor used. On many types of construction the costs generated

by the equipment are about as high as the costs involved in employing labor. The costs generated by the equipment are of two kinds—operating expense and depreciation. Job charges on account of the operation of equipment depend on the period during which the equipment is used. The same is at least largely true of charges on account of depreciation.

From this it naturally follows that if the design of the construction organization is correct, the proper amount of equipment is supplied to the job. In this case the basic charge for equipment—that is, the rate at which equipment charges will accrue—is correct. Thereafter, the gross amount of these charges will depend on the working period. In short, the conditions which hold the use of labor at a minimum also hold the use of equipment and the charges resulting from this use at a minimum. On jobs where a considerable amount of equipment must be used, the saving here, which results from efficient job management, may be as important as the saving in the labor field.

Hand-labor economies

There is one other matter which deserves a word of comment in this connection. Anything man can build at all can be built by hand labor equipped with little or nothing but hand tools. It follows as a matter of logic that the use of machinery and equipment is a matter of expediency rather than a matter of necessity. Therefore, the extent to which machinery and equipment should replace labor equipped with hand tools or with the somewhat heavier tools in the use of which animal power is required always is a matter deserving some consideration by those who are designing construction organizations. In the recent past labor costs have been relatively high, and the tendency has been to use power-driven equipment to replace as much of it as possible. In many instances the replacement has been of obvious advantage from the cost standpoint. But in rather a surprising number of cases there is reason to believe that replacements were due to an attitude of mind among contractors and among engineers, and to sales promotion rather than to any close analysis of costs.

There is no need to go into this matter at length here, but it may perhaps be proper to remark that the need of the day is for opportunities for men to earn their living. On this account, if for no other reason, those who are designing construction organizations should give more attention to the extent to which human labor can be used to advantage. In doing this it will naturally occur to those responsible for organization design that cheap labor extends the scope of the field in which it can be used to advantage. But in addition to this, the use of labor also offers the advantage of a reduced investment in fixed assets. It would be undesirable to lay too much stress on these matters, for, quite obviously, it would be undesirable to place construction altogether on a hand-labor basis. The fact remains, however, that in a good many operations close analysis will show that from the cost standpoint the use of equipment is optional. In such cases, while so many need work, such operations should be handled on a hand-labor basis.

Previous articles in this series were published on July 20, 1933, p. 73; Aug. 3, p. 129; Aug. 17, p. 193; Aug. 31, p. 255; Oct. 5, p. 407; Oct. 19, p. 473; Nov. 2, p. 531; March 8, 1934, p. 319, and April 12, p. 474.

Letters to the Editor

Fuel Oil and Gasoline Tanks

Sir—The design of fuel-oil tanks has assumed considerable importance of late, as fuel oil is very generally displacing coal as a fuel. Heretofore, however, insufficient attention has been given to earth pressure and live loads in the design of such underground tanks, to conform with recent theory and practice on the subject of earth pressure. A method of tank design has been developed by the writer in connection with tank installations in New York City that may be applied tentatively pending experimental verification. The suggestions of municipal engineers as to practice in their localities are invited.

The most common type of under ground fuel-oil tank used in New York is of 550-gal. capacity, and is 4 ft. in diameter by about 6 ft. long. For such tanks the rules require a tank at least ¼ in. in thickness. But when the tanks are buried 2 ft. below ground and heavy trucking passes over them, wheel loads should be taken into consideration and the thickness increased or a reinforced-concrete slab placed over the tank to take a 24,000-lb. wheel load at its center (as required by the New York City building code), distributed at an angle depending on the nature of the soil. For earth pressure the method of pipe design as given in Hool and Johnson's "Concrete Engineers' Handbook" may be used. There, experimental values for earth pressure at varying levels and trench widths are given. The pipe is designed for moment, considering a unit length of the pipe as a beam having a length equal to the pipe diameter. The moment at the center will be $WL/16$, where W is the total load on the pipe for a unit along its length. The heads of the tank will act to reduce the moment, so that for tanks the moment may be reduced as for a slab supported on four sides, or the moment may arbitrarily be reduced one half, giving $WL/32$.

In these formulas the restraining action of soil at the sides of the tank is not relied upon, and only the pressure of the earth on the top of the tank which deflects it a maximum at the center of its width is considered, though the customary one-third of the vertical pressure acting horizontally may be deducted. Tank steel is figured at 11,000 lb. per sq.in. safe unit stress. By the method described, a tank 7 ft. below the ground would have to be ⅜ in. thick instead of the ¼ in. allowed by the rules for a 4-ft. diameter tank.

Gasoline tanks in New York are inclosed in 12 in. of concrete except at the top. For either soil pressure or wheel loads, encasing the tank in concrete also at the top will result in protecting it by four concrete arches (considering a transverse section through the tank and protection) whose side thrusts are counteracted by each other. Or, a reinforced-concrete slab may be placed over the tank to distribute the wheel-load pressures over a wider area. It should be stated in this connection that in excavating for tanks and placing them the wall and column stresses acting (in case of sand) at 60 deg. from the vertical should not pass through tank or excavation unless slab protection or increased tank thickness is resorted to or the walls or columns are underpinned.

Tanks should also be designed for internal pressure as for water pipes. When filled by pressure and the liquid may reach into the vent pipe, the hydrostatic head is the distance from the top of the vent pipe to the bottom of the tank.

MORRIS M. BERMAN.

Brooklyn, N. Y.
Feb. 26, 1934.

Side Pressure on Retaining Walls

Sir—Referring to the second article by Professor Terzaghi on large retaining-wall tests (*ENR*, Feb. 22, 1934, p. 259), the conclusions by the author regarding the distribution of side pressure prior to slip would be hard to reconcile with a rational design.

Taking as an example a double-deck subway structure with water-flushed backfill and offering no chances for a slip, a distribution in accordance with Fig. 4 of the author's article would give an approximately trapezoidal pressure diagram for the upper wall and a curved one for the lower, with a minimum intensity at the bottom, as indicated by a solid line in Fig. 1. If the lower struc-

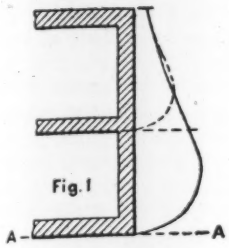

Fig. 1

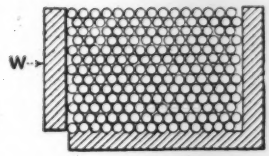

Fig. 2

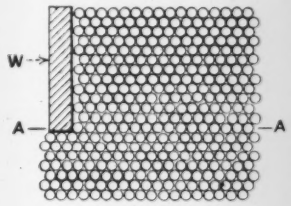

Fig. 3

ture were omitted, however, the pressure diagram for the upper wall would change as shown by a dotted line. It does not appear likely that the presence or absence of the lower structure should affect to such an extent the pressure distribution on the upper wall.

A feature that seems to have been overlooked in the author's tests may give a clue to this inconsistency. In actual earthwork the backfill is resting on an elastic stratum of similar material. The side pressure at plane *A-A* may be considered due to forces set up by wedge action as the individual grains of the backfill are being forced, through the weight of the overlying material, into the gaps between the grains of the supporting ground surface. Due to the lateral flexibility of the support, these small and irregular forces are integrated over a considerable width and transmitted to the bottom of the wall.

This is brought out in Fig. 3, the grains being replaced here by elastic spheres. The wedge action set up by the weight of the spheres above layer *A-A* tends to effect a crowding of the underlying spheres toward the wall *w*, and through the elastic yielding of the spheres below a pressure against the bottom of the wall results.

In the author's test arrangements, this bottom pressure is given no opportunity to develop, the yielding ground being replaced by a solid slab of practically unyielding material, the friction of the sand against the slab prohibiting any lateral displacement of the bottom layer. This prevents the integration and transmission of the forces referred to—not only of those created at the surface of the slab, but more or less also of those acting in horizontal planes directy above, because of the shallowness and consequent low elasticity of the intervening layers of supporting material.

The condition is represented by Fig. 2, showing the spheres of the bottom layer cradled so as to restrain them from lateral movement. The forces resulting from the crowding tendency cannot reach the wall *w* but produce only a horizontal pull between the slab and the opposite wall of the bin.

The shielding influence thus exerted by the bottom slab may account for the upward displacement of the side-pressure resultant, as this seems to be due not so much to any peculiar distribution of pressure as to the elimination of pressure at the bottom portion of the wall.

A closer approximation to practical conditions might be had if the bin were extended downward for some distance below the movable wall, or if the bottom of the bin were covered by a layer of clay.

Minimum pressure intensity at the bottom of the wall may conceivably occur where the wall is resting on rock. It may likewise be expected for a limited time where excavation has been made in compacted material, when arch action may be temporarily set up in the backfill. To depend on the relieving effect of such arch action on permanent structures, however, as J. C. Meem proposes, does not seem to be a safe proposition.

ANDERS BULL,
Assistant Engineer,
Board of Transportation.

New York, N. Y.
March 22, 1934.

Bridge-Pier Construction with Artificial Sand Islands

Sir—In your March 29 issue, on the contents page under the title "Coming Articles," there appears this statement: "Inadequacy of the usual caisson-sinking methods for the record-size piers required in the West Channel of the San Francisco-Oakland Bridge led to the development of a new procedure: depth to rock necessitated the open-dredging process, but the horizontal dimensions of the foundations made sand islands impracticable; and the tidal flow as well as the nature of the soil rendered false bottoms for the caissons too hazardous."

I am in a position to question these broad statements. My particular interest is centered in the so-called "sand island" method of lowering caissons. I originated the use of sand islands, which make use of structural material in forming artificial islands, and I designed the islands used at the Suisun Bay Bridge (Martinez-Benecia Bridge) on the Southern Pacific Railway, and those used at the bridge now being constructed at New Orleans. Pier No. 1 at New Orleans is in 85 ft. of water at the time of writing, with water at 10-ft. stage, and is designed for 95 ft. Pier No. 6 of the San Francisco-Oakland Bridge is 105 to 112 ft. deep. I made preliminary designs for the use of "sand shell islands" for the San Francisco-Oakland Bridge, and they are practicable, safe and economical. As a matter of fact, I have preliminary designs for a sand island in 150 ft. of water with dimensions sufficient to handle the caissons of the San Francisco-Oakland Bridge. There is nothing impracticable about them.

So far as the air-controlled caisson is concerned, it is new in application on a large scale, but the idea is not new. Four patents, two of which have expired, cover this type of construction for bridge piers. One of the expired patents was obtained for the construction of the San Francisco-Oakland Bridge.

I agree that false bottoms for the large caissons would be hazardous. The sand-island method cannot be so classed. It is adequate, safe and economical.

M. F. CLEMENTS,
Bridge Engineer,
Northern Pacific Ry. Co.

St. Paul, Minn.,
April 6, 1934.

Peg Adjustment of Levels

Sir—The "Simplified Peg Method for Adjusting Engineer's Level" to which you devote some space on p. 253 of your issue of Feb. 22 is not only *not* simplified but is positively wrong. That is, the peg method should be used for adjusting the cross-hair rather than the bubble, and the method given is even more complicated than the old cumbersome method still seen occasionally in textbooks.

The axis of the bubble should be made perpendicular to the vertical axis of the instrument and then left alone. Otherwise time will be wasted every time the telescope is turned to a new position, whereas if the bubble stays centered short sights and readings to tenths over fairly long distances can be taken without the necessity of touching the leveling screws.

Having the bubble thus adjusted, the simplest method of making the so-called peg adjustment (i.e., making the line of sight parallel to the axis of the bubble) is to determine the *correct* difference of elevation of two points by taking readings from approximately the midpoint between them; and then to use this correct difference of elevation to find the correct rod reading for the distant point, having set up near one of the points and taken a reading on it. The cross-hair is then adjusted to give this correct rod reading on the distant point, with the bubble centered as usual.

Thus it is unnecessary to measure any distances, nor is it necessary to make the second set-up so close to the near point as to require sighting through the objective of the telescope, though if the instrument is badly out it might be desirable to take a second reading on the near point for a check.

Yours for straight thinking and real simple and direct methods whenever they are correct and applicable.

F. A. BARNES,
Professor of Railroad Engineering,
Cornell University

Ithaca, N. Y.,
March 3, 1934.

ENGINEERING NEWS-RECORD

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Regional Code Representation

LOCAL administrative committees representing the divisional code authorities set up under the approved chapters of the Construction Industry Code are to be appointed during the next few weeks. Meanwhile the several trade associations have been asked to appoint temporary representatives as advisers to the state directors of the National Emergency Council in the 53 cities where the state directors are located. These representatives will form local committees having no administrative powers; their functions are purely advisory, to promote an understanding of the codes. They can make no rulings or interpretations of the codes, nor can they take up matters relating to employment. This should be clearly understood if confusion is to be avoided. The divisional code authorities are the only bodies authorized to make rulings on the codes. Similarly, care should be exercised in registering contracts and paying fees to unauthorized parties. In the case of the general contractors division, the Associated General Contractors of America has been made the agent for the registration of work undertaken. Others, especially those seeking to collect a fee, should be shunned. If "rackets" needlessly preying on the industry are to be avoided, the limitations on the local representatives of the code authorities as outlined in our news pages need to be fully understood.

Returning Solvency

ONLY a definite improvement in the field of municipal financing can account for the increasing number of requests from towns and cities asking that their PWA loans and grants be reduced to include only the thirty per cent grant. During the past two weeks these requests have exceeded 35 in number, and the result has been to release $9,000,000 of PWA funds, which were immediately re-allotted to projects that had been left on the waiting list. This reduced pressure on the federal financing of local public works has its corollary in last week's highest municipal bond sales for more than a year. Every instance in which PWA assistance for a non-federal project is reduced to a straight grant means that the spread of the fund is tripled. Nevertheless the aggregate figure of this financing released for re-allotment probably will not bulk large when compared to other PWA figures. The real significance of the current development is that it indicates healthier municipal credit, which in turn is further evidence of the breaking of the economic clouds.

Spring Road Hazards

THE KILLING of three persons on the Storm King Highway a week ago by a fall of rock serves to emphasize the need for constant vigilance if reasonable safety is to be obtained on mountain highways. Rock slides are bound to occur, especially in spring, but they may occur on the calmest day of summer. In the day of the slow-moving horse-drawn vehicle they were of little consequence; roads were less deeply cut into the face of the landscape, traffic was less dense, and the driver generally heard the fall in time to avoid it or saw it in time to stop. Today, with closed high-speed automobiles, the situation is different, and the highway maintainer must learn the lesson that railroad maintenance men learned years ago. Through the use of protective nets, through patrolling and through scaling of loosened rocks they have almost eliminated a frequent cause of accidents in earlier years. Frequent scaling is especially productive of results, as the loosening of rocks generally is progressive and dangerous masses usually can be pried loose and rolled down harmlessly. In the case of scenic highways such as Storm King the best thing to do may be to close off the road to all but necessary traffic when frost is coming out of the ground or after hard storms.

For Long-Time Gain

ACTIVE PROSECUTION of demolition work in many cities brings visible proof of the need for clean-up and modernization of many obsolete elements of our communities. The need has come to be appreciated particularly through the scrutiny of building defects that resulted from the depression, a closer scrutiny than ever before carried out systematically. The timely opportunity for useful employment which demolition affords is by no means its only value, for in addition to this present gain a long-time gain of even greater importance may be realized. The groundwork can be laid for better building codes and health and safety regulations that will encourage more efficient and more economical building. Studied in the light of public interest, both relaxation and further restriction of building practices may be warranted by the teachings of building obsolescence derived in course of demolition work. By thorough present restudy of codes, building materials and methods it will be possible to lead forward in direct line from the demolition movement into a period of sounder reconstruction of cities for the future.

CCC'S Birthday

A YEAR AGO THIS WEEK the Civilian Conservation Corps was breaking ground for its first camp. After a comparatively short inaugural fanfare this early element of the recovery program has been doing its work in quiet effective manner, keeping thousands of young men from becoming disgruntled unemployed and advancing the forest and soil-conservation program of this country at many times its normal speed. The original plan of the President to take young men from the streets and provide them with healthy outdoor work has not been materially altered, and today more than 300,000 youths are serving voluntarily in the forest improvement army. Partly as a result of what this force did, the forest-fire loss of the nation last year was only 17 per cent of the annual average. In addition, permanent improvement has been effected on many miles of forest roads, national forests and parks have been cleaned up, and extensive work has been done on erosion control and flood prevention. To the United States Army much credit is due for its efficient work in mobilizing and caring for this organization and to the Forest Service for planning and supervising the constructive work of most of the camps. In all, the CCC program can be classed as one of the best-executed and most effective undertakings of the depression relief schedule.

Programming Future Work

FEDERAL Relief Administrator Hopkins last week threw the light of cold fact upon the obscure statistics of recovery in reporting that relief activities reached an all-time peak in March, with 4,700,000 families on the list. The increase over earlier months does not mean that there are more unemployed but rather that the exhaustion of their resources has put more unemployed persons into the relief class than were reclaimed from this class by the NRA and public-works re-employment. This fact alone gives the impressive picture of the havoc wrought by industrial disruption, and in doing so it lays powerful emphasis on the urgent importance of continuing the re-employment campaign.

It is now known that little more can be contributed to re-employment by the industries and trades concerned with commercial goods. The decisive part that the durable-goods industries play in employment has only recently been given practical recognition by the organization of the NRA's special committee to promote the revival of these industries. Some concrete plan was hoped for from this committee that would hasten recovery in the durable-goods field with its many millions of workers, but beyond recommending that banking facilities be provided for financing the intermediate-term credits required by industry and that the Securities Act be revised to facilitate new capital flotations, the committee has been unable so far to point out any road to revival other than intensive prosecution of public construction.

That there are vast potential activities in this field of public construction is proved beyond question by the four billions of dollars in projects accumulated in the Public Works Administration office after the recovery fund was exhausted. These projects, moreover, do not include many important types of improvements which the safety and comfort of the people require; no grade-crossing elimination is included, for example. In short, the nation's improvement work is only begun.

Introduction of the La Follette public-works bill in the U. S. Senate last week is the most definite expression that has yet been heard of the re-employment need and of the opportunity for construction through which it may be met. With its total of ten billion dollars as a three-year fund for a continued construction recovery campaign, this bill is viewed by some as a fantastic exaggeration, but in actual fact it is anything but fantastic. It distorts neither the facts of unemployment nor the opportunities for useful construction. Its total is based on estimates of existing physical needs that were compiled by a group of engineers and construction advisors called in conference by the author of the bill. The bill therefore is a fair yardstick by which may be gaged the effectiveness of any alternative recovery proposals. Mr. Hopkins' relief statistics indeed give it the sensational status of being the only adequately proportioned recovery plan that has been advanced.

Obviously, both for the long-time program and for the more immediate problem of next winter it is necessary to make timely preparations. The four billions of project plans in the PWA files are only a beginning toward an effective program. They represent the most pressing improvements, but in doing so they raise further important questions. What improvements are next in order? Are plans and legal preliminaries prepared to enable these further projects to be put in application form quickly? Has study been made on which a continued public construction program can be based with due regard for efficiency and economy? Without practical answer to such questions the attack on unemployment and economic debility is unlikely to progress far.

Understanding the Code

INQUIRIES and reports indicate that much confusion still exists as to the operation and scope of the construction code. This is true even though the code is quite clear and understandable. Though the code digest and catechism that we published two weeks ago* revealed some weaknesses and ambiguous points that remain to be cleared up by rulings or amendments, most of the doubts can be resolved by reference to the terms of the code itself.

Three primary points should become firmly fixed in the mind of everyone who has to do with construction. *First*, the intent and purpose of the code is to establish fair competitive practices within the construction industry, and in that industry alone; the code cannot govern other industries. *Second*, the code applies only to members of the industry, which means employers, not employees. *Third*, the spirit of the code is to make the square deal prevail throughout construction, and in this light all provisions of the code may be better understood. Further, square-deal principles afford the safest interpretation of such points as are not explicitly covered in the code; it is not entirely unlikely that the code authority's later rulings and interpretations will apply these principles to all questions that arise in application.

Important doubts remain as to the position of manufacturers of materials and "standard products" under the code, and how their quotations or "bids" should be regarded. The code expressly excludes these manufacturers from the class of subcontractors; they are not members of the construction industry except when they assemble or install their products at the site or when they furnish products specifically designed for a particular construction project. Chiseling by contractors in respect to prices of standard products and materials must be taken care of mainly by the codes of these manufacturing industries, though the Construction Code Authority may yet extend the meaning of "bids" to binding price quotations made by materials men in response to specific inquiries by contractors and subcontractors.

The construction code is, after all, a set of self-government regulations. There is nothing mysterious or complex about it. If the members of the construction industry know the code and govern themselves accordingly, they will have no difficulty in carrying out its provisions. It is important, however, to remember that every general construction contract of $2,000 or more must be registered and a fee paid, based on the size of the contract; failure to register and pay the fee (under an executive order of the past week) is a punishable violation of the code; and unless a contractor includes the amount of fee in his bid he will penalize himself for the amount.

An intimate knowledge of the various provisions is essential to a full understanding of the meaning and purposes of the code. The code has been written and placed in force. A full and complete regulation of the construction industry on a fair and sound basis will come only after experience of code operation has provided the opportunity for interpretations and changes.

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*Reprints of the digest and catechism of the code are available in pamphlet form at the office of *Engineering News-Record*. Copies of Chapters I and II of the construction code may be secured from the General Contractors' Divisional Code Authority, Munsey Bldg., Washington, D. C.

CURRENT NEWS

Administration Bill For Public Works Is About Ready

PRESIDENT ROOSEVELT is preparing to submit to Congress a budget of $1,500,000,000 for "recovery" expenses, including public works and direct relief, during the fiscal year commencing July 1. This will be well within his $2,000,000,000 estimate of last December, according to reports, and part of the total has already been anticipated by other bills, and will provide only $500,000,000 for public works.

Distribution of the fund will depend on circumstances. At present, it appears that direct relief will receive the majority because at the closing up of the CWA, April 1, Administrator Hopkins had left about $450,000,000, and relief needs during the current month will take $90,000,000. The FERA administrator hopes this is a peak, that seasonal improvement in business plus employment created by the PWA program and NRA code limitations will reduce the number of relief rolls, from the present figure of 4,700,000 families—16,900,000 individuals—including those on work relief. It is apparent, however, that before another winter sets in, relief funds will be nearing exhaustion.

To extend the public works money as far as possible, Jesse Jones, chairman of RFC, will discuss with the President, the proposal by which RFC would participate in financing public works through loan of funds needed on non-federal projects over and above the 30 per cent PWA grant. During the past year PWA has financed 100 per cent of the cost of all projects that cities could not finance in the open market.

Chairman Jones said this week that without an increase by Congress in the limit on its borrowing capacity, the amount that the RFC could apportion for public works would "not be a lot in comparison with requirements."

The RFC is not committed to the plan which PWA Administrator Ickes strongly endorses but the decision rests with the President. Ickes would have the RFC take off his hands bonds purchased during the past year against projects for which PWA allotments have been made, as well as bonds purchased for projects which will receive grants out of the next appropriation for public works.

Ground Water Fluctuations to be Studied in Prairie States

An investigation of the ground water fluctuations in the seven prairie states will be undertaken by Prof. Howard E. Simpson, University of North Dakota and state geologist, appointed to make the study under the general supervision of the Mississippi Valley Committee. This study is a part of the general investigation of rainfall and runoff for which $25,000 was recently allotted by the Public Works Administration.

Low Bid on Fort Peck Dam Diversion Tunnel Is $7,311,940

The low bid for the construction of the four diversion tunnels for the Fort Peck Dam project in eastern Montana was submitted jointly by Silas Mason, Inc., New York, and Walsh Construction Co., Davenport, Iowa, at a figure of $7,311,940 when bids were opened at the Kansas City division office of the U. S. Engineers, April 13. Work will consist of driving and lining four 28-ft. diameter diversion tunnels, each about one mile long, through the shale formation of the Fort Peck dam site. The project is located about sixteen miles south of Glasgow, Mont., in the eastern part of the state. The present preliminary work was described *ENR*, April 5, 1934, p. 444.

Record-Breaking Concreting Continues at Boulder Dam

Concrete placing on the Boulder dam project continues at a record-breaking rate with 224,000 cu.yd. placed in the dam during March, and a total of 260,000 cu.yd. placed on all phases of the project during the same period. This output followed the placing of 192,000 cu.yd. of concrete in the dam during February and a total plant output for that month of 213,000 cu.yd. Last October was the first month in which the concrete output on the project exceeded the 200,000-cu.yd. per month figure.

Twin City Sewage Project Drops Plan to Carry Own Insurance

After an extended controversy relative to the carrying of insurance during the construction of the $18,000,000 Twin City sewage project, the matter was settled when the board of trustees, April 9, voted to strike out provisions for blanket insurance coverage from the specifications. Accordingly, compensation, public liability and property damage insurance will be left for the individual contractors, as well as the question of surety bonding. The proposed plan, which was eliminated by this recent action of the trustees, would have provided for the trustees of the district carrying all of the insurance with all of the contractors for the work under one blanket policy. The rate set by the rating bureau of the Mirnesota Compensation Insurance Board was $18.60 per 100 dollars of payroll for compensation coverage as it would be carried by the trustees, while the rate established by the board for individual contractors on the same work was $14.82.

The board also decided to advertise for composite bids on the three first contracts and bids will be received for these three jobs separately or as a unit. This work will consist of the construction of two interceptors and the sinking of one 11 ft. shaft 80 ft. deep at an estimated cost of about $1,000,000. All of the present work is situated in the eastern section of St. Paul.

Michigan Sewage Works Operators Meet For Short Course and Discussions

FOLLOWING a three-day short course in laboratory practice of sewage analysis and plant hydraulics, a two-day program of papers and discussions was held at Lansing, Mich., at the Tenth Annual Conference of the Michigan Sewage Works Association. Seventy attended the conference and 25 registered for the short course which was given by professors of the Michigan State College.

J. A. Volk presented a description of the Traverse City sewage treatment works on Lake Michigan, which has now completed a full year's operation. A notable achievement has been the reduction in pollution of Little Traverse Bay as indicated by the results of frequent bacteriological surveys. James R. Rumsey reported on the experience at the Grand Rapids plant where small gas engines were run on sewage gas.

A paper by N. G. Damoose reported the results being obtained on the Grand River by the sewage treatment plant at Grand Rapids. A complete series of graphs showed the variation of D.O., B.O.D. and bacteria at several sampling stations above and below the treatment plant. These data were compared with a series of determinations made before the plant was constructed and indicates the degree of improvement. Thomas Doyle related the experiences at Pontiac from the plant operators' viewpoint. Particularly interesting was the comparison of the hit or miss methods used in operating a plant during his first years of operation as compared to the scientific control of later years.

W. L. Mallmann, Michigan State College, discussed data obtained from chlorinating sewage at Flint and Traverse City. The use of a sodium thiosulphate sampling bottle apparently would give a more nearly correct picture of the effectiveness of chlorination. George Hubbell, Hubbell, Hartgering & Roth, discussed the effect of inlet arrangements on sedimentation tanks. He brought out an interesting comparison of various types of inlets on a model tank constructed in accordance with the old Dearborn Imhoff tank plans. As a result of the studies, a new type of inlet was developed and installed with a marked increase in sedimentation efficiency of the tanks.

At the annual business meeting Pierre Welch, Flint, was elected president, J. A. Volk, Traverse City, vice-president, E. C. Miller, Dearborn, director, and W. F. Shephard, secretary and treasurer. The conference voted to endorse the adoption of rules and regulations of the Michigan Department of Health for certification of plant operators.

New $10,000,000,000 Public Works Bill Before Congress

FORMAL introduction by Senator LaFollette of the $10,000,000,000 public works bill which he has had in preparation for a month will force the issue which the Administration so far has avoided. Dwarfing by comparison any probable Administration measure the bill is all that the advocates of a huge public-works program could hope for. On the other hand it ignores the disposition in Congress to appropriate only for specific projects. The bill provides for allotting the $10,000,000,000, within broad limits, as follows:

(a) A minimum of $5,855,000,000 for non-federal projects, exclusive of other allocations, with no limit on the proportion of the cost assumed by the federal government and a proviso that existing terms with respect to loans "be liberally construed, insofar as the requirement of security for loans made is concerned, to the end that the public works program contemplated hereby shall be expedited to the fullest extent possible." The present law limits the grant on non-federal projects to 30 per cent of the cost of labor and materials and requires reasonable security for loans.

(b) A minimum of $1,850,000,000 as grants to the states for highway construction, of which not less than $1,250,000,000 shall be allocated for elimination of hazards to highway traffic referred to in Section 204-a-1 of the present law, such as grade separation at crossings, reconstruction of existing railroad grade crossing structures, and relocation of highways to eliminate railroad crossings. The LaFollette bill specifically includes track elevation and depression through cities, but does not change the prohibition of use of funds for acquiring land, rights of way or easements in connection with railroad grade-crossing elimination projects.

(c) A minimum of $125,000,000 for federal-aid road construction when matched by the states in accordance with the Federal Highway Act of 1921.

(d) A minimum of $50,000,000 for roads in national forests, national parks, Indian reservations, public lands, Alaska, Canal Zone, Puerto Rico and Virgin Islands.

(e) A minimum of $1,500,000,000 for new building construction, with a maximum of $175,000,000 for federal buildings, a minimum of $500,000,000 to the Home Owners Loan Corporation for new home and building construction financed by first mortgages at low interest rates, and a minimum of $825,000,000 for loans and grants to finance non-federal public buildings, low-cost housing and slum clearance projects, hospitals and, as specifically included by amendment, school buildings approved by the U. S. Commissioner of Education.

(f) A minimum of $20,000,000 for the Coast & Geodetic and Geological Surveys.

(g) A minimum of $600,000,000 for direct relief.

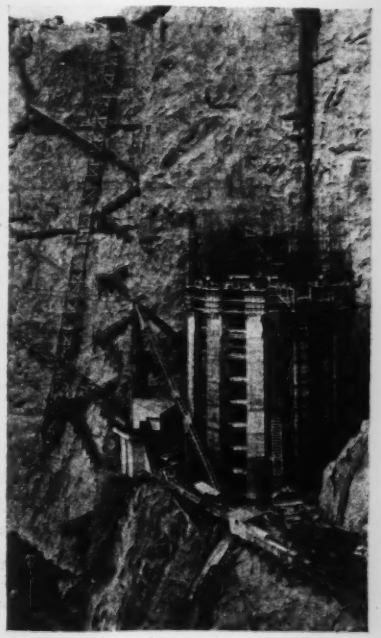
No funds out of the new appropriation would be available for naval construction. The life of the PWA would be extended for another year beyond June, 1935, to carry through the tremendous program. The cost would be met by Treasury borrowing.

The bill was referred to the Committee on Education and Labor but LaFollette may start action by offering it as an amendment to one of the pending appropriation bills. Otherwise he cannot bring his bill up, as the power to originate appropriations rests in the House.

Speculation centers on what the Administration will do. The rank and file in Congress, particularly the House, are not so much interested in a grand total as in its distribution. The House appropriations committee is waiting for the estimates of the budget bureau. If these are submitted for appropriation in a lump sum, as was this year's appropriation of $3,300,000,000, the Administration will have to resist a drive from members of Congress to pin allotments to specific projects.

Several bills proposing the expenditure of huge sums for public works preceded the LaFollette bill, but none of them made any progress toward enactment except the Cartwright bill appropriating $400,000,000 as grants to states for continuing emergency highway construction programs. Favorably reported by the House Committee on Roads, March 23, the Cartwright bill is pending on the House calendar and probably will be overtaken by the Administration's public works bill when introduced (see news report on p. 518).

The Ellenbogen bill would also appropriate $400,000,000 for continuing the road program and provides in addition $2,000,000,000 for other public works. Several amendments have also been offered to change the present law.



BOULDER DAM INTAKE TOWER

One of the four intake towers at Boulder Dam shown about 80 ft. high or one-fourth of its final height. Discharge from the reservoir through these intakes will be delivered by penstock tunnels to the power plant below the dam. This is the downstream tower on the Nevada side and the surface of the concrete in the dam is shown in the extreme lower left-hand corner. The stiffleg derrick beside the tower is used to transfer the concrete buckets from railway to cableway.

PWA to Start Condemnation of Land to Speed Slum Clearing

Court proceedings will be initiated to acquire land for a federal housing project in Atlanta, Ga., to expedite the start on active construction. Administrator Ickes has requested the attorney general to file condemnation proceedings in the Atlanta federal court for the acquiring of about 140 parcels of land, required for the construction of the University Housing Project, a $2,100,000 housing operation for negroes in that city, which has strong local endorsement.

Although this is the first time when condemnation proceedings have been instituted, the Administrator announced that representatives of the housing corporation have been active in a number of other cities, making appraisals and taking options on land. The condemnation suit is not being used, according to the Administrator, in the form of a threat but merely to speed the Atlanta housing program. The government, he said, will pay a fair price and continue to consider reasonable offers even after legal proceedings have been started.

Administrator Ickes said: "It has been decided to exercise the right of the Government to institute condemnation proceedings to acquire land at reasonable prices as a fair and expeditious method of carrying forward the low-cost housing and slum-clearance program.

Important Changes in Engineers To Be Made by the Army in June

Several important changes in division and district officers of the Corps of Engineers have been announced by the War Department. The moves, which are effective in June, include:

Col. Thomas H. Jackson, division engineer, South Atlantic district, has been appointed North Atlantic division engineer effective June 5. He will succeed Col. George M. Hoffman, present North Atlantic division engineer who will retire June 15, 1934. The post of South Atlantic division engineer has been filled with the appointment of Col. Earl I. Brown, who is at present with the Board of Engineers for Rivers and Harbors, Washington. This appointment becomes effective on or about June 1, 1934.

Lt. Col. Ulysses S. Grant, now a student at the Army War College, Washington, has been assigned to 1st Engineers, Fort Dupont, Del., as soon as his present course is completed. Lt. Col. John J. Kingman has been named Boston district engineer effective July 1. He is now Providence (R. I.), district engineer.

Major Mason J. Young, assistant Philadelphia district engineer, has been appointed to fill the post of Providence (R. I.) district engineer. Major Donald H. Connolly, instructor at the General Staff School, Fort Leavenworth, has been appointed Chicago district engineer effective July 1. Major John F. Conklin, now a student at the Army War College, Washington, has been appointed Huntington (W. Va.) district engineer, effective July 2. Major Raymond G. Moses, now a student at the Army Industrial College, Washington, has been appointed assistant to the Lower Mississippi Valley division engineer at Vicksburg, Miss., effective July 1.

First Year of CCC Closes with 320,000 at Work in 1,468 Camps

On the first anniversary of the Civilian Conservation Corps, this feature of President Roosevelt's recovery program is employing 320,000 young men in 1,468 camps located in every state in the Union. On April 6, 1933, the organization was created and orders issued to establish the forest camps. The first 300 men enrolled in the CCC two days later, and the first camp was established near Luray, Va., April 17. During the twelve months' period this organization has provided work for about 600,000 young men between the ages of 18 and 25, in camps operated by the U. S. Army. During the year, these men have sent home to dependents about $60,000,000 as deductions from their $30 a month pay and Secretary Dern estimates that two-thirds of the actual disbursements of $255,000,000 has gone directly into the channels of industry, commerce and agriculture.

The majority of the work of the corps has been done in the national forest areas and in federal and state parks. Some work has been carried on in privately owned forest areas. Partly as a result of this organization, the forest fire loss in the national forests during the 1933 season was $325,000, which is only 17 per cent of the average annual loss. Much permanent improvement work has been carried out in the national forest areas including the construction of roads, trails, telephone lines, lookout-stations and fire breaks. The corps also planted 72,000,000 trees. Flood control and erosion prevention work by the corps has been beneficial to 500,000 acres, mostly in the form of check dams and revegetation. In addition, the work of the CCC has aided materially in making the national and state parks accessible and useful as a result of brushing and clearing. Seed collecting and nursery work will permit a larger reforestation work in 1934.

Waterworks Operating Problems are Discussed at Indiana Meeting

FOLLOWING a three-session school for water superintendents and filter operators, the Indiana Section of the American Water Works Association held its 27th annual convention, April 12, at Purdue University, Lafayette, Ind. Private wells, valve insertions, flushing practice, stream cleansing, responsibilities of waterworks officials, accounting, meter research and PWA progress were the principal waterworks topics covered. Malcolm Pirnie, president, A.W.W.A., analyzed the value of an adequate supply of water for fire fighting purposes on the day following the convention, when the waterworks officials met with the Indiana fire chiefs in three joint sessions.

Charles Streithof, Evansville, stated that in 1933 his department inserted a large number of both large and small valves in the Evansville system without shutting off the water. The system was installed in 1873, when finances were low. It was surveyed in 1914 and was found to have valves spaced 1500 to 3600 ft. apart. In 1933 this spacing was reduced to 350 to 750 ft. The time required to insert a 30-in. valve, exclusive of excavation and backfill, with a crew of 5 men was 16 hours. For 20- and 24-in. valves the time was from 8 to 10 hours. Three 6- or 8-in. valves could be cut in per day. Examination of some of the cut out sections showed that the pipe was half full of sediment and rust. This year it is proposed to clean the whole system.

A symposium on flushing brought out that the Indianapolis Water Co. carries out its flushing operations according to a schedule starting April 1 each year. The whole 700-mile system is gone over progressively, starting from the pumping stations. Formerly two crews took about 2½ months to complete the program, the work being done during the day. Two years ago, said Charles Winkle, it was found that a better job could be done at night. There is less trouble with traffic and less interference with pumping station operation. Three crews work six nights a week. The flushing frees the pipes of accumulated iron rust. It has been found that as much as 1½ to 2 in. of soft iron oxide settles in the large mains in 9 or 10 months. During the flushing program every hydrant is tested. Dead ends are flushed to eliminate tastes and odors, some every two weeks, others every four weeks, according to the length of the dead end and the number of customers connected to it.

Howard Dill, of Richmond, reported that the community flushes its water system in the fall and spring. Care is taken to notify factories, particularly where the water is used for process purposes. W. H. Durbin, of Terre Haute, said two 45-deg. ells were used where the hydrant set back of the curb to spray the water as it issued to prevent erosion. Others used a pipe to extend to the curb.

The relative economy of using private wells or a city supply was analyzed by John A. Bruhn, who has investigated all of the cost factors for 60 rock wells in Indianapolis. They include such factors as the original investment for drilling and casing, pump equipment, connections to the distribution system, approved cross-connection to the city system for standby service and housing. The cost of failures and the subsequent repairs, on examination, disclosed some unusual factors seldom thought of. The short life of a casing is due to alternate wet and dry conditions, galvanic action, crooked-pipe troubles and clogged screens. The type of pump has much to do with operating cost, capacity and economy. First high cost often shows the best economy. The drop of the ground water levels under pumping at Indianapolis is about 2 ft. per year and the loss in capacity and efficiency is a serious problem. The average cost of an overhaul, involving removal, dismantling, repair and replacement, is $300. The usual quality troubles with well water were noted.

In discussing this topic L. R. Howson, Chicago, cited the 500 wells in Fond du Lac and the epidemic that resulted from a dropping water elevation. L. A. Geupel, state sanitary engineer, asked the support of the members in connection with tourist camp wells.

The Indiana Department of Conservation has undertaken to clean up the streams of the state. W. H. Frazier, sanitary engineer, exhibited a map which indicates that practically all the streams of the state are badly polluted. Because of this, he said, Indiana loses recreation traffic to Michigan and other states. Industry wants to cooperate, in his opinion, if it is advised what to do. In 10 years, said L. A. Geupel, damage suits brought because of stream pollution have increased from $250,000 to $1,000,000 annually.

The newly elected officers are as follows: President, C. C. Foutz, vice-president, C. K. Calvert; secretary-treasurer, Bernard H. Jeup, Indianapolis. H. S. Morse is national director.

MISSOURI RIVER BRIDGE AT KANSAS CITY APPROACHING COMPLETION

Present indications are that the steel superstructure of the Missouri River bridge at Kansas City will be completed by May 1. A private toll structure, owned by the Regional Bridge Co. and financed by the RFC, the new bridge forms an integral part of the new Seventh Street Trafficway in Kansas City, Kan. (*ENR*, Oct. 9, 1933, p. 459).

The substructure consists of four piers about 100 ft. deep, constructed by the pneumatic method, and ten piers on pile foundations. The superstructure consists of a 574-ft. approach deck truss, two 300-ft. simple through-truss spans and a cantilever bridge involving a 447-ft. center span and 417-ft. anchor spans. The view is looking south toward the Kansas side. The bridge is being built by the Kansas City Bridge Co., to plans prepared by Sverdrup & Parcel, consulting engineers, St. Louis.

PWA Continues to Transfer Funds to New Projects

AS A RESULT of requests for the reduction of previously awarded loans and grants to 30 per cent grants and the rescission of several other awards, the PWA has been able to re-allocate about $2,800,000 to new non-federal projects which had previously been on the waiting list.

Of the 15 communities which have been able to sell bonds in the open market and thereby reduce their demand on PWA financing to a 30 per cent grant, five are located in the state of Kansas, two in Massachusetts, two in Rhode Island, two in Missouri and one each in Colorado, Minnesota, Texas and Wisconsin. The types of projects include a wide range covering water supply improvement, sewage disposal plants, school construction, airport improvement and street work. Two $1,000,000 projects are represented, the extension of the Boston municipal sewer system and the construction of sewers and a disposal plant for Springfield, Mo.

Three PWA allotments were withdrawn during the week, including a grant of $15,000 to Thief River Falls, Minn., which was rescinded because no effort had been made to carry forward the project, and two allotments, one of $140,000 to Greenville, N. C., for hospital construction and the other a minor $2,000 loan and grant withdrawn because the recipients notified PWA that the projects had been reconsidered and they did not desire to go forward with them at this time. The total of $157,000 released from these three projects has already been re-allotted to waiting non-federal applications.

As a result of these releases of PWA funds, Administrator Ickes has announced allotments for 67 more non-federal construction projects located in 28 states. These allotments are listed in the accompanying table.

Razing 17 Tenements Starts New York Drive on Slums

With the acceptance of a bid of $2,770 for the demolition of seventeen privately-owned tenements, the New York City drive to clear its slum areas of firetrap and insanitary tenement buildings has reached the active stage. In addition to the present initial project, owners of 140 more tenements have offered their property for demolition, according to an announcement by Langdon W. Post, tenement house commissioner. Under the present agreement the city tears down the building and permits a reduction of 25 per cent in the current tax assessment. Many of the sites will be used temporarily for playgrounds.

About twenty bids were submitted for the razing of the first group of buildings. The work will be done by men hired by the city's welfare department working under the supervision of tenement house inspectors. Bids for similar work on other sites will be called in a few days.

Federal House Modernization Program to be Released Soon

Announcement of the administrators' housing program outlined in last week's issue (*ENR.*, April 12, 1934, p. 478) is expected within a week. Repair and modernization loans up to $2,000 will be included for all types of structures and not restricted to residences. Mortgage loans on new construction will be confined to dwellings, although a large part of construction during the coming year is expected to be for industrial and commercial buildings. This program is to be financed through federal reserve intermediate credit. House mortgage loans will be discounted by the Federal Home Loan Banks out of the proceeds from sales of bonds by Home Owners Loan Association, which has been authorized to issue $2,000,000,000 of credit. This program, according to reports, is meeting opposition from the large insurance companies.

Code Authority Appoints Temporary Representatives

TO PROMOTE understanding of the Construction Code and to assist the various divisions of the industry locally, the construction code authority has requested each national trade association sponsoring the code to appoint a temporary representative in each of the fifty-three cities where state directors of the National Emergency Council are located. These temporary representatives will form a temporary coordinating committee to disseminate correct information about the construction code and the method of administration thereunder, and to advise state directors and others as called upon. The committees will have no administrative powers or responsibilities. It is understood that they will be set up within a period of two or three weeks.

Local Committees

Local administrative committees possessing administrative powers and duties will be appointed only by the divisional code authority established under each approved chapter of the construction code. For instance, by the general contractors, master painters, elevator manufacturers, cement-gun contractors, tile contractors, and others as chapters are approved. The majority of the state and local administrative committees for the general contractors code administration will be appointed within a week or ten days' time.

These local administrative committees or agencies officially delegated by the respective divisional code authorities will deal only with trade practices or other matters solely affecting employers and will have nothing to do with labor rates or labor conditions. As mutual or area agreements to fix standards of hours of labor or labor rates in a particular division are negotiated and approved under the procedure prescribed in Article III of Chapter

DETAILS OF PWA LOANS AND GRANTS FOR NON-FEDERAL WORK

Locality	Loan and Grant or Grant	Type	Amount
Cassville, Wis.	G	Sewers	$16,000
Chambersburg, Pa.	G	Turbo generator	2,100
Zanesville, Ohio	L. & G	Sewer system	166,000
Kearny County, Kan.	L. & G	Court house and jail	60,000
Garfield County, Colo.	G	Hospital bldg.	8,300
Salisbury, Pa.	G	School bldg.	6,500
Chandler, Okla.	L. & G	Power plant	142,000
Alma, Mich.	L. & G	Sewer system	100,000
Denver, Colo.	G	Sewers	24,000
Talladega County, Ala.	G	Court House repairs	2,900
Talladega County, Ala.	G	Jail	16,500
Moss Point, Miss.	L. & G	Bridge	18,500
Muhlenberg County, Ky.	L. & G	Road	175,000
Worcester, Mass.	L. & G	Hospital	250,000
New Castle County, Del.	G	School	10,000
Pushmataha County, Okla.	L. & G	Court House	69,750
Franconia, N. H.	L. & G	Waterworks	100,000
Arp, Tex.	L. & G	Waterworks	26,000
Elizabethtown, Ill.	L. & G	Waterworks	38,000
Independence, Mo.	G	School	74,400
Madawaska, Me.	L. & G	School	91,100
Glenwood, Ark.	L. & G	Sewers	32,000
Concho County, Tex.	L. & G	Bridge	9,000
Jefferson, Mo.	G	Sewer system	1,700
Mt. Vernon, Mo.	G	Waterworks	2,800
Elizabethtown, N. C.	L. & G	Waterworks	36,000
Austin, Minn.	G	Electric plant	46,000
Riley County, Kan.	G	Highway bridges	4,500
Riley County, Kan.	G	Highway bridge	3,300
Mt. Vernon, Mo.	G	Diesel engine	6,900
Latexo, Texas	G	School	1,300
Cimarron County, Okla.	L. & G	School	32,500
Andover, Conn.	G	Road construction	12,600
Catawba County, N. C.	L. & G	School	43,000
Warren County, Pa.	L. & G	School	$30,200
Prague, Okla.	G	School	3,600
Kermit, W. Va.	L. & G	Waterworks	65,000
Mt. Pleasant, S. C.	L. & G	Waterworks	75,000
New Eagle, Pa.	L. & G	School	20,200
Blackwell, Okla.	L. & G	School	224,500
Kansas City, Kan.	G	Street repairs	15,500
Kansas City, Kan.	G	Street repairs	21,600
Ramapo, N. Y.	L. & G	School	50,000
Henry, Ill.	G	Street repairs	20,500
Center Township, Mo.	G	Road repair	1,200
Black Hawk County, Ia.	G	School bldg.	10,000
Denton County, Tex.	L. & G	School bldg.	24,500
Prairie View, Tex.	G	College bldgs.	1,800
Burlington, Ia.	L. & G	Armory	34,000
Chester, Conn.	G	Road construction	13,800
Camden County, N. C.	G	School bldg.	8,800
Cape Elizabeth, Me.	G	Fire station	1,500
Findlay, Ill.	L. & G	Waterworks system	50,000
N. Wilkesboro, N. C.	L. & G	Waterworks	59,000
Crown Point, Ind.	L. & G	Waterworks system	129,000
Society Hill, S. C.	L. & G	Waterworks system	24,000
Decatur County, Ia.	G	School bldg.	500
Thompson, Conn.	G	Road construction	42,200
Austin County, Tex.	G	School bldg.	1,000
Nashville, Tenn.	L. & G	Street repairs	200,000
Owatonna, Minn.	G	Street construction	2,900
Owatonna, Minn.	G	Road construction	2,300
Thief River Falls, Minn.	G	Electric plant	4,800
Kiowa County, Okla.	G	Court House	11,700
Orono, Me.	L. & G	School	129,400
Onondaga County, N. Y.	L. & G	School	154,000
New Britain, Conn.	G	School	7,600
		Total	$3,068,750

I of the Code, special employer-employee boards will be appointed to handle matters of compliance. Such mutual or area agreements, if approved by the President, will affect every employer in the particular division of the industry performing the type of operations covered by the mutual agreement. Employers bound by existing labor agreements which affect only the signatories thereto will continue to be bound during the life of such agreements or until these agreements are superseded by other agreements.

It is important that employers guard against collection of fees for registration of construction work or services not made by duly authorized code representatives. This registration will be carried on by each divisional code authority or its designated agency. In the case of general contractors, the Associated General Contractors of America is the agent for registration of work undertaken. This organization has appointed about a hundred agents throughout the country. While no evidence has appeared of any effort by unauthorized persons to collect fees under this registration, employers should be on guard to prevent racketeering.

Upholds Registration

On April 14 the President issued an executive order upholding compulsory assessment of the members of any industry to support the administration of its code, and authorizing the code authority to institute legal proceedings, if necessary, for collection of such assessments. It also restricts the right to participate in the selection of the code authority or to make use of any emblem or insignia of the NRA to members of the industry complying with the code and contributing to the expense of its administration.

An administrative order by General Johnson supplementing the executive order provides a procedure for enforcing an equitable contribution to support the code administration. Under this procedure the Administrator must approve an itemized budget of estimated expenses and an equitable basis of contribution, and the code authority must give the member of the industry due notice of how and when contributions are payable. Code authorities, generally, have felt the need of this official approval of assessment of administrative expenses.

Colorado Will Use Convicts on Highways Again After 11 Years

For the first time in twelve years convict labor will be used for the improvement of Colorado highways, it has been decided by Gov. Edwin C. Johnson. Colorado was the first state to use convicts in highway building, about 25 years ago. The governor said that prisoners will not replace outside laborers, as the convicts will be employed on projects for which no funds for paying civilian workers have been made available. The first project will be on the highway between Canon City and Salida, just west of the Royal Gorge of the Arkansas river.

"I expect to be severely criticized by labor for adopting a policy of putting convicts on highway work," Governor Johnson said, "but it must be understood that this will be done only in a limited area."

Steel Arch of 1,100-ft. Span To Be Built in Africa

Construction of the Birchenough Bridge across the Sabi River, in Rhodesia, is planned for the immediate future, with a scheduled completion date of April, 1935. The bridge is to be of steel arch type with a 1,100-ft. span and will give access to the road leading through the ranching lands of Victoria to Gwelo and Bulawayo. At present the Melsetter district is cut off for eight months of the year from direct communication with towns lying to the west.

The bridge will cary a double line of road traffic and sidewalks. The designer of the structure is Ralph Freeman, who also designed the Sydney Harbor Bridge. The span will be constructed of chromodore steel.

The bridge is named for Sir Henry Birchenough, president of the British South Africa Co. The firm of Dorman, Long & Co., Ltd., has secured the contract for the construction work, and the engineers will arrive from Great Britain in April. The bridge will cost $630,000.

River Rouge Negotiates With Detroit Over Water Service

Officials of River Rouge, Mich., who have on file an application for PWA funds to build an independent $360,000 waterworks system, are now negotiating with Detroit for a reduction in rates from 60c. to 40c. per 1,000 cu.ft. If the lower rate is granted the PWA application will probably be dropped, but the Detroit Water Board is at present making a restudy of its suburban business. It furnishes an average of 3.6 m.g.d. to twenty suburbs at a cost of $820,000 annually. A metropolitan water system has long been considered and the demands of River Rouge and several other towns adjacent to Detroit which also have PWA applications may further this move. The contract with River Rouge dates from 1900 and provides that all pipes and other structures which form part of the distribution system shall become the property of the Detroit Water Board because there is an area beyond River Rouge which receives Detroit water through River Rouge pipes. This provision in all the suburban contracts would simplify the formation of a metropolitan waterworks system.

SOCIETY CALENDAR

AMERICAN SOCIETY FOR TESTING MATERIALS, Atlantic City, N. J., June 25-29, 1934.

AMERICAN SOCIETY OF CIVIL ENGINEERS, annual convention, Vancouver, B. C., Canada, July 11-14.

AMERICAN WATER WORKS ASSOCIATION, annual convention, New York, June 4-8, 1934.

AMERICAN WELDING SOCIETY, annual meeting, New York, April 26.

INTERNATIONAL CITY MANAGERS' ASSOCIATION, annual conference, St. Louis, Oct. 15-17.

SMOKE PREVENTION ASSOCIATION, annual convention, June 19-22, Buffalo, N. Y.

NEW ENGLAND WATER WORKS ASSOCIATION will hold its spring meeting at Greenwich, Conn., May 1.

MARYLAND-DELAWARE WATER AND SEWERAGE ASSOCIATION will hold its eighth annual conference at Washington, D. C., May 3-4.

Boston Tunnel Extension Would Co-ordinate Transit Lines

Proposals are under discussion by the city authorities, the Boston Elevated Railway and the Boston, Revere Beach & Lynn Railroad for a co-ordination of service facilities in East Boston by a 4,500-ft. extension of the East Boston tunnel from Maverick Square to Day Square at an approximate cost of $3,000,000. The Boston Elevated Railway, through chairman Henry I. Harriman of its board of trustees, informed Mayor Mansfield on Jan. 23 that the road is willing to advocate such an extension of the tunnel in order to establish rapid transit service between the downtown district and a proposed terminal station of the Revere Beach road (the Narrow Gauge line) near Day Square. The plan contemplates the interchange of passenger traffic between the two companies on an equitable basis of fare sharing, and the ulitmate abandonment of the cross-harbor ferry service which has been a "bottleneck" on the Narrow Gauge system for generations. Mayor Mansfield has stated that the project must be self-liquidating in order to receive his approval. If the plan goes through the construction of the tunnel extension will probably be done by the Boston Transit Department. The Revere Beach road has suggested establishing a terminal nearer the East Boston airport, with facilities for interchanging traffic with the Elevated.

Court Decision Permits Start on $7,000,000 Baton Rouge Bridge

Bids will be called within 30 days for construction of the combined rail-highway traffic bridge across the Mississippi River at Baton Rouge, with opening 30 days later, according to an announcement of John Lyle Harrington, consulting engineer, Kansas City, following a decision of the Louisiana supreme court confirming the right of the Louisiana Highway Commission to sell bonds for the structure. The supreme court decision resulted from a friendly suit undertaken at the instance of New York bond attorneys to test the right of the commission to contruct a bridge with railway facilities for lease to the carriers. This right was affirmed by court opinion rendered March 26.

Field surveys and borings at the bridge site, north of the Standard Oil Co. refinery, two miles above Baton Rouge, have been completed and plans are being prepared in the office of Harrington and Cortelyou, Kansas City. The bridge will carry a single railroad track and two 18-ft. roadways. The estimated cost is $7,000,000 of which 30 per cent will be met by direct grant and the balance by PWA purchase of state highway bonds. Contracts for carrier use will be signed soon following a conference of A. P. Tugwell, commission chairman, with representatives of the PWA, the Louisiana and Arkansas Railroad, the Gulf Coast Lines, and the Missouri Pacific System.

In addition, authority for advertisement of sale of $500,000 of state highway bonds has been granted the Louisiana Highway Commission by the Louisiana state highway advisory board. The bonds will carry five per cent interest and will be from the major highway and bridge program issue already available.

Personals

FRED S. CHILDS, for the past ten years principal assistant engineer to the late George A. Johnson, has entered private consulting engineering practice in New York City.

JAMES J. FORRESTEL has assumed office as director of the building division, department of public works, Buffalo, N. Y., suceeding Fred O. Francis, resigned.

RICHARD ORTH has been appointed division engineer of District No. 2 of the Ohio highway department. He has been in the bridge department of the same organization for the past nine years.

GEORGE W. SCHUSLER is now superintendent of the Bureau of Sanitation, Department of Public Health, Pittsburgh, Pa.

ARTHUR D. WESTON, chief engineer and director of the sanitary engineering division of the Massachusetts Department of Public Health, has been elected vice-president of the Boston Society of Civil Engineers.

ARTHUR V. SHERIDAN, engineer of design, Bureau of the Bronx, has been appointed to the position of chief engineer of the bureau.

COLONEL J. W. WOODRUFF, Corps of Engineers, district engineer, Philadelphia, Pa., has recently been detailed a member of the board of engineers for rivers and harbors.

MAJOR FRED W. HERMAN, head of the Huntington (W. Va.) district, U. S. Engineers, has been transferred to Washington, D. C. and F. B. Duis, principal engineer of the same district, has been transferred to the district office at Cincinnati, Ohio.

R. E. HAWLEY, engineer of design, Department of Plant and Structures, City of New York, has retired after 36 years of active service in the department. His retirement will start some time in April.

PROF. MELVIN L. ENGER, head of the Department of Theoretical and Applied Mechanics, University of Illinois, since 1926 and a member of the faculty since 1907, has been named dean of the College of Engineering and director of the Engineering Experiment Station replacing Dr. A. C. Willard, recently named president of the University.

PROF. FRED S. SEELY, a member of the University of Illinois faculty since 1909, has been made head of the Department of Theoretical and Applied Mechanics to fill the vacancy caused by the election of Prof. M. L. Enger to the deanship of the College of Engineering.

JOHN D. SPINKS, associate civil engineer, Corps of Engineers, is now supervising construction of Lock and Dam No. 3 on the Cape Fear River, North Carolina. This assignment follows Mr. Spinks' work in the Treasury Department supervising construction of a Marine hospital at Mobile, Ala., completed late in 1933.

CONSTRUCTION STATISTICS OF THE WEEK

HEAVY public-works awards bring this week's engineering construction total up to $32,248,000, the second highest week of this year. Public awards are the highest for the year—$30,051,000, of which federal contracts total $12,780,000 and state and municipal $17,271,000.

Public buildings and "unclassified" projects are responsible for this gain, the former with $11,146,000 under contract and the latter $10,075,000. Highways dropped back to $5,500,000 for the week.

Large awards include: post office, Houston, Tex., $750,000; high school, Rochester, N. Y., $934,000; school, Buffalo, N. Y., $558,000; auditorium, Kansas City, Mo., $4,228,000; complete camp at Fort Peck dam site, Mont., $1,454,000; two electric substations, Fort Peck Dam, $732,000. The largest highway letting was by Tennessee, $834,000, while Oregon awarded the most bridge work, $685,000. In the large "unclassified" group the Fort Peck tunnels went at $7,312,000, the Fort Peck transmission line at $1,171,000, and the Delaware River Joint Commission high-speed transmission line, $1,374,000.

New capital totals $10,655,000 this week, of which $310,000 is the net increase of PWA allotments over rescissions and changes. $7,320,000 is accounted for by state and municipal bond sales, $2,744,000 by corporate issues and $250,000 by PWA allotments to federal departments.

CONTRACTS

(Thousands of Dollars)

	Weekly Average Apr. 1933	Prev. 4 Weeks	Week Apr. 19 1934
Federal Government	$1,756	$4,144	$12,780
State and municipal	5,060	13,543	17,271
Total public	$6,817	$17,687	$30,051
Total private	5,531	4,792	2,197
Week's total	$12,348	$22,479	$32,248
Cumulative to date: 1933	$251,777	1934	$380,253

NEW PRODUCTIVE CAPITAL

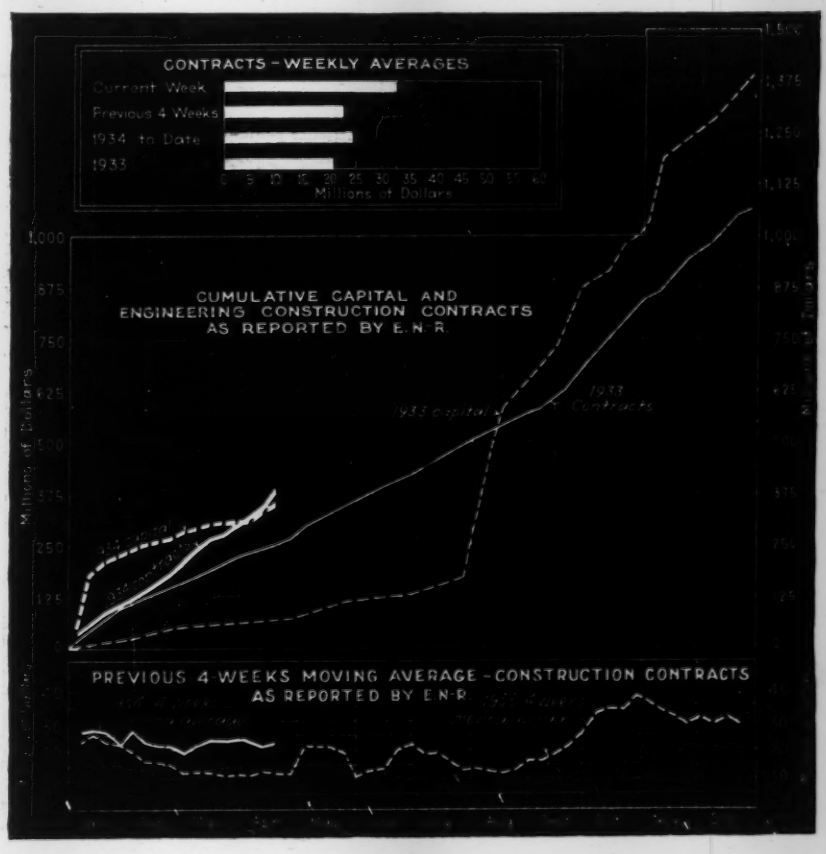
(Thousands of Dollars)

1934	Week Apr. 19	Cumulative
State and municipal	$7,320	$94,436
PWA allotments, S&M	311	196,806
Corporate issues	2,774	13,369
PWA allotments, private	....	45,131
Total, Non-Federal	$10,405	$349,742
PWA allotments Federal Constr.	250	48,392
Total new capital	$10,655	$398,134
Cumulative to date: 1933	$72,201	1934 $349,742
Rescinded or reduced allotments (from 1933 PWA totals)		$43,323

NOTE: These figures include private bonds, and stock sold for productive purposes; state and municipal bonds for construction; PWA loans and grants to states and municipalities, including the special highway funds; PWA private loans, and allotments for Federal construction.

INDEX NUMBER

E.N.-R.-Cost		E.N.-R.-Volume	
April, 1934	195.86	March, 1934	108
March, 1934	194.04	February, 1934	88
April, 1933	160.16	March, 1933	62
1933 (Average)	170.18	1933 (Average)	102
1932 (Average)	156.97	1932 (Average)	127
1931 (Average)	181.35	1931 (Average)	220
1913 (Average)	100		


CONTRACTS - WEEKLY AVERAGES
Current Week
Previous 4 Weeks
1934 to Date
1933
Millions of Dollars
CUMULATIVE CAPITAL AND ENGINEERING CONSTRUCTION CONTRACTS AS REPORTED BY E.N.R.
Millions of Dollars
PREVIOUS 4 WEEKS MOVING AVERAGE - CONSTRUCTION CONTRACTS AS REPORTED BY E.N.R.

Construction Equipment and Materials

New Road Machine Removes Grader's Windrow

The Hi-Way windrow eliminator, recently announced by the Hi-Way Service Corp., Milwaukee, Wis., is an attachment for any motor patrol grader that eliminates the windrow deposited by the grader blade in the middle of the road.

In the operation of the windrow eliminator, hydraulic lifts are provided for raising and lowering the blade. This blade is not a grading blade, and is only to be used for smooth spreading of the surplus material back across the graded surface and depositing stones and clods at the roadside. The blade is reversible so that the windrow may be taken from either side.

Hi-Way Windrow Eliminator

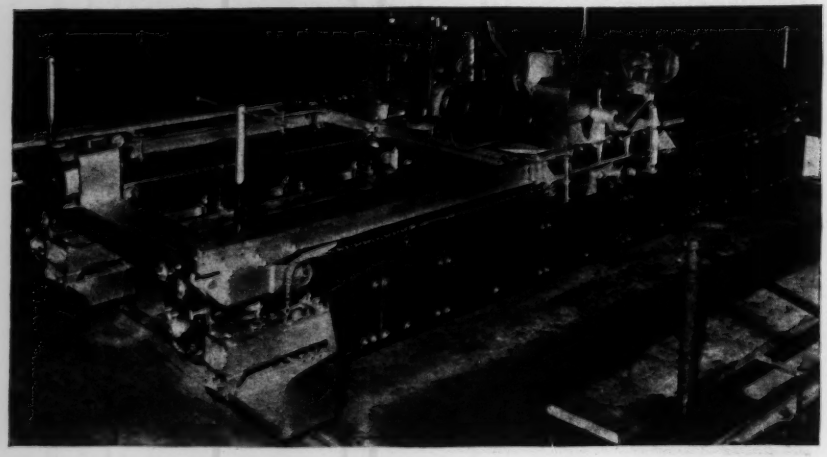
New Road-Finishing Machine Powered by Electricity

The Blaw-Knox Co., Pittsburgh, Pa., has developed a road finisher for concrete and bituminous paving which is fully electrically powered, power being furnished by a generator directly connected to a gasoline engine, mounted on the machine. The manufacturer states that before this machine was placed on the market it was tested and proved on four paving projects, totaling 20 miles, in which it proved its ability to build ribbon-smooth roads speedily and economically.

The machine has four speeds forward and reverse, instead of two, which was standard in the past. Traction is now equal on all four wheels.

Briefly summed up, the chief characteristics of this new machine are as follows: it has faster forward and reverse motion; less time is required to change machine widths; it is faster in crown adjustment; it has greater traction; raising jacks for elevating and lowering the finisher above the forms are built into the machine. In addition the machine is provided with electric connections for floodlighting for night work. It is designed for the addition of vibrating attachments, tamper, and accessories for bituminous paving.

Road-finishing machine by Blaw-Knox

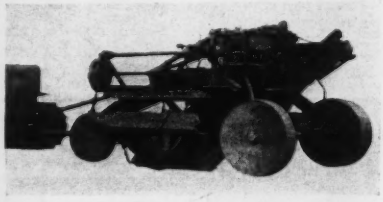


Power-Controlled Grader Uses Power Take-off on Tractor

Great operating economy, extreme simplicity, lower first cost, less upkeep and full capacity under all but the most severe operating conditions, are some of the advantages claimed for the new power take-off, power controlled elevating grader recently announced by Caterpillar Tractor Co., Peoria, Ill.

The new model features power operated controls; a 48-in. shaft-driven carrier, wide drum-type wheels, extensible axles, inclosed gear drives, and wheels, drive shafts, drums and belt rollers mounted on ball or roller bearings. Power is supplied to the carrier, operating controls and pan cleaner by a rear power take-off on the tractor.

The operator has an unobstructed view of the furrow, the belt, the wagons and the tractor driver. The new machine may be equipped with 19, 22, or 25-ft. carriers.



Caterpillar power-controlled elevating grader with power take-off

New Equipment in Brief

Pile Hammers. A new line of pile hammers, designated the B-3 type, has been introduced by the McKiernan-Terry Corp., New York, N. Y. Heavier ram weights are used in the new design and structural improvements have been made.

Brake Lining. Wagner CoMaX, a new brake lining, has been announced by the Wagner Electric Corp., St. Louis, Mo. It is described as non-compressible under pressure. It has been developed especially for high-speed operation.

Gasoline Engines. The Hercules Motors Corp., Canton, Ohio, has announced the ZX series of gasoline engines and power units. Two models are included, developing 22 and 24½ hp., respectively, at 4,000 r.p.m.

Wood Preservative. The Tennessee Eastman Corp., Kingsport, Tenn., has introduced a new wood preservative which is called No-D-K. It is made from creosote oil extracted from wood and recovered from the company's wood alcohol process. The maker states that the new material is not caustic and will not burn the skin.

Service Trucks. The Ditwiler Mfg. Co., Galion, Ohio, has announced the Redi-Service truck, which is equipped with the Ditwiler power take-off, on which can be mounted a wide variety of equipment to be driven by the truck engine. The unit is equipped with an all-steel canopy express body with an all-steel top. It is designed to carry five men in addition to the mounted equipment. The maker claims that a wide variety of equipment can be used in conjunction with the unit to utilize the engine power. A standard unit developed for highway and general utility work includes an air compressor, an arc welder and a centrifugal pump.

Business Notes

THE REO MOTOR CAR CO. announces that A. L. Struble has been appointed sales manager of the Truck Division.

UNION ASPHALTS & ROADOILS, INC., a new firm dealing in paving asphalts and asphalt products, has opened an office in the Business Men's Assurance Building, Kansas City, Mo.

JOSEPH T. RYERSON & SON, INC., Chicago, Ill., announces that Harold B. Ressler, vice-president in charge of sales, will move to New York to take charge of the company's Jersey City plant.

JOHNSON-MARCH CORP., announce the appointment of Gloucester P. Hevenor as vice-president and general manager; and the removal of its offices from Long Island City to 52 Vanderbilt Ave., New York City.

NATIONAL FIREPROOFING CORP., Pittsburgh, Pa., announces that L. M. Christie, vice-president in charge of clay conduit sales, is now directing the sales of all products of the company.

COLPROVIA ROADS, INC., has announced that its executive offices are being moved from New York City to Rochester, N. Y., Lincoln Alliance Bank Building. Charles W. Denniston has recently been elected president of the company.

NATIONAL GYPSUM CO., Buffalo, N. Y., has announced that they have purchased the metal lath plant of the Kalman Steel Corp. at Niles, Ohio. The company states that the property will be converted into a fully modern plant for the manufacture of metal lath and metal reinforcing.

PORTLAND CEMENT ASSOCIATION, Chicago, Ill., announces that C. A. Clark, for the past four years president of Clark & Burrows, Inc., of Dallas, Texas, has rejoined its staff. Prior to entering the construction equipment business, Mr. Clark was for 13 years with the Portland Cement Association as field engineer, district engineer, and finally as regional manager in charge of the Southwestern territory.